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PREFACE.

With the rapid increase in the development of the water resources of the United States there has arisen among capitalists and engineers throughout the country a great demand for information in regard to the flow of streams. Although much has been written on the methods of measuring stream flow and the interpretation of the data, such information is widely scattered through periodicals and Government reports, many of which are out of print and therefore not easily accessible for use by either the student or the engineer. The short descriptions of stream gaging in text-books are indefinite in character, stating only general methods and giving but little information in regard to the details of field work or the conditions requisite for reliable records of river discharge.

Experience with the graduates of many of the best engineering schools in the country indicates that these men have generally had but little instruction in hydraulic field work or methods, and are practically helpless in attempting to carry on even the simplest hydrologic investigation. Correspondence with engineers in all sections of the country shows that they are not getting the maximum benefit from the available stream-gaging data, apparently on account of lack of understanding of the records.

In the preparation of this book there has been brought together from all available sources information in regard to the best practice in this work. Much new matter is also presented, especially the descriptions of the conditions necessary for good gaging stations at which measurements of discharge may be made either by weir, current meters, floats, or slope; the routine of the selection, establishment, and maintenance of gaging stations; the details of the field work of discharge measurements, and the office methods of computing the regimen of flow.

The authors hope and believe that the information here presented will be valuable both to the student and the engineer.

Acknowledgments are here made to the United States Geological Survey, the United States Weather Bureau, and the American Society of Civil Engineers, for use of cuts and other material; also to Messrs. J. C. Stevens, R. H. Bolster, G. M. Wood, F. W. Hanna, and E. C. Murphy for assistance and suggestions.

JOHN C. HOYT.
NATHAN C. GROVER.

WASHINGTON, D. C., *May*, 1907.

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ILLUSTRATIONS.

RIVER DISCHARGE.

By John C. Hoyt and Nathan C. Grover.

CHAPTER I.

INTRODUCTION.

HISTORICAL SKETCH.

Practical acquaintance with and useful application of the general laws of flowing water date from the first century. In A. D. 98 Rome was supplied with water by nine aqueducts having an aggregate length of 250 miles and discharging 27,000,000 cubic feet a day. Yet hydraulics was not regarded as a science until about the fourteenth century, and there was little advancement until the seventeenth century, when, owing to the influence of Galileo, more rapid progress was made. The principal investigations during the seventeenth and the first half of the eighteenth century were made by Castelli (1628), Torricelli (1643), Guglielmini (1700), Pitot (1730), and Bernouilli (1738), and the work done was mainly theoretical.

Active experimental hydraulic investigations were begun by Professor Michelotti in 1764, and from this time the modern school of hydraulics dates. Writings and investigations made prior to 1764 are now of comparatively little importance to the practicing engineer.[a]

In 1775 M. Chezy, the celebrated French engineer, developed the formula now known by his name, $V = c\sqrt{Rs}$, in which V = velocity and c = a coefficient combining the effects of roughness of the bed and all other conditions affecting velocity except the slope (s) and hydraulic radius (R), which equals the area of the cross-section of water divided by the wetted perimeter. This was the first algebraic expression of the law of moving water and has served as the basis of all subsequent slope formulas.

[a] A detailed review of early hydraulic studies is given in "Physics and Hydraulics of the Mississippi," by Humphreys and Abbot.

In the United States attention was first given to the flow of water in open channels between 1840 and 1850, in work on the Mississippi River and its tributaries. In 1850 Humphreys and Abbot started their extensive investigations on that river. In 1855 Francis published the results of his investigations made at Lowell, Mass., in which he developed his formula for flow over weirs. In 1870 Ellis, in his work on the Connecticut River, added much valuable data. The studies of Humphreys and Abbot and of Ellis, and other work of the engineers of the United States Army have, however, been confined to special investigations. It was not until 1888, when the United States Geological Survey began to collect data in regard to the water supply of the country at large, that the general applicability of hydraulic laws was investigated and methods were developed for determining the regimen or the distribution of flow.

In starting the hydrographic work of the Survey, Major J. W. Powell, then Director, stated:[a]

It will be necessary to gage a certain number of representative streams at all seasons of the year, so as to ascertain their total discharge and its seasonal distribution, and also to gage a greater number of streams at certain seasons determined to be critical.

Starting with this object, the Survey developed methods for universal stream gaging and collected data in regard to the flow of streams in all sections of the United States, which are now extensively used by engineers in enterprises involving the use of water. In all this work the Survey has contended that, inasmuch as the flow of a stream is constantly changing, data of reasonable accuracy showing the distribution of flow over several consecutive years are of more importance than very accurate measurements covering short periods of time.

SCOPE OF DISCUSSION.

The hydraulic engineer is interested in water from the time it reaches the earth in the form of rain or snow until it returns again to the atmosphere in the form of an invisible vapor. Of the water which falls upon the earth, a portion immediately returns to the atmosphere; a portion soaks into the earth, reappearing in vegetation or as surface water, or remaining below in small amount as permanent ground water; and another portion stays for a time on the surface of the earth, in streams ponds, lakes, or oceans. A knowledge of the phenomena that pertain to these changes in conditions and of the physical and chemical prop-

[a] Tenth Ann. Report U. S. Geol. Survey, 1890, p. 8.

erties of the water itself constitutes the science of hydrology. Every feature of this great science is of direct value in the economic development of the country, but probably none is of greater importance than a knowledge of the discharge of surface streams and of the conditions that affect its magnitude and variations.

In this discussion of surface flow the following phases of the subject have been considered: The methods of measuring and computing stream flow; the laws that govern such measurements and the degree of accuracy obtainable; the phenomena that affect the flow of the streams; and the uses to which the data are applicable.

OUTLINE OF METHODS.

The discharge of a stream is the quantity of water flowing past a given section in a unit of time and may be obtained as the product of two factors—(1) the area of cross-section, which depends on the shape and dimensions of the bed and banks and on the stage; (2) the velocity, which depends on the surface slope, the roughness of the bed and banks, the hydraulic radius, and the channel conditions along the stream. In general these governing factors are controlled by the stage. Therefore the discharge may be considered as a function of the stage.

By means of this general law it is possible, from discharge measurements covering the range of stage, to construct a rating curve and table from which, the mean daily stage of the stream being known, the daily discharge can be taken. This daily discharge is essential in all investigations where the quantity of flowing water is an important factor, as from it the total annual flow and the distribution by days, months, and seasons can be determined. In such investigations isolated observations of stage or discharge are of little value unless they are made at stages that are known to be extreme; and even then the record of the duration is equal in importance to that of the magnitude of the flow itself.

The discharge of a stream is expressed in various units, among which the second-foot is the most common. This term is an abbreviation for cubic foot per second, which is equivalent to the quantity of water flowing in a stream 1 foot wide, 1 foot deep, at a velocity of 1 foot per second. The determination of the discharge is termed "discharge measurement," and points at which discharge measurements are made and records of the daily fluctuations of stage are kept for determining the daily flow are termed "gaging stations." These stations may be grouped in two classes, one comprising those where measurements are

made by the velocity-area method, which consists in measuring the velocity of the current and the area of the cross-section; the other comprising those where measurements are made by the weir method, in which the discharge is obtained by measuring the head on a weir and using a weir formula.

The selection of a gaging station and the method to be used in determining the discharge depend upon many factors and are accomplished in various ways. These governing factors are the funds available, the period of time over which the observations can be extended, and the conditions of the stream to be measured, as explained in the following pages.

CHAPTER II.

CONDITIONS AFFECTING STREAM FLOW.

Water, in its ceaseless round from atmosphere to earth and return, in its courses over or through the land or through vegetation, affords an endless number of interesting problems for study. In this discussion, which pertains essentially to stream flow, there will be considered only the conditions affecting the quantity and distribution of water from the time it reaches the earth in some form of precipitation until it flows into the ocean or is returned to the atmosphere. The amount and distribution of the water in the streams, which is derived primarily from precipitation, are modified by evaporation, temperature, geology, topography, vegetation, and artificial control. Since the effects of these influencing factors can not as a rule be differentiated from one another, no data can in general be given to show their magnitude; tendencies only can be discussed.

It is important to observe that while precipitation and evaporation largely affect the distribution of the run-off, they also practically control its total amount. The other factors, except in so far as they affect precipitation and evaporation, exert their influence principally on the distribution of flow and but slightly on the total quantity of discharge.

PRECIPITATION.

All the water that appears in streams has at some time been condensed and precipitated from the atmosphere. The quantity, intensity, and distribution of precipitation are therefore prime factors influencing the quantity and distribution of the run-off.

Rain gages for measuring precipitation consist of a collecting cylinder, which exposes a circular surface for collecting the rainfall, and a storage vessel in which the water is retained until measured.

The standard rain gage of the United States Weather Bureau (fig. 1 and Pl. I, A) is best suited for the measurement of precipitation under ordinary conditions. This gage consists of a receiver, an overflow attachment, and a measuring tube.

In this rain gage the exposed surface is 8 inches in diameter and is

connected by means of a reducing funnel with the measuring tube, which has an inside area of cross-section of $\frac{1}{10}$ the area of the surface of the receiver. The measured depth of water in this tube is therefore 10 times the depth of precipitation.

Other rain gages are arranged for weighing the collected waters and for easily reducing the observed weights to inches of depth.

Rain gages are usually non-recording and must be visited daily whenever precipitation takes place. The attendant measures the precipitation either directly, by measuring the depth, or indirectly, by weighing the water which has been collected since his last record, empties the gage, and replaces it for further use. Satisfactory recording rain gages that require only occasional attention and that are therefore suited to the collection of records in uninhabited and mountainous sections have not yet been devised.

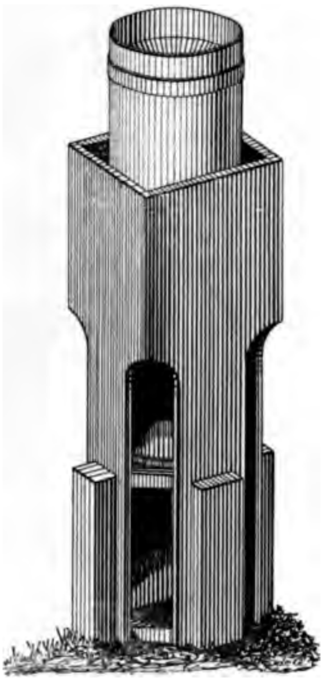


Fig. 1.—Rain Gage and Support.

Inaccuracies in precipitation records may result from one of two causes—(1) poor exposure of the gage or (2) incorrect collections and measurement of snowfall.

Rain gages in slightly different positions, if badly exposed, catch very different amounts of rainfall. Two gages placed within a few yards of each other may show a difference of 20 per cent in rainfall in a heavy rainstorm. Wind seems to be the principal factor in producing this difference. The stronger the wind the greater the difference is likely to be. In a high location eddies of wind produced by walls of buildings divert rain that would otherwise fall in the gage. A gage near the edge of the roof, on the windward side of a building, shows less rainfall than one in the center of the roof. The vertical ascending current along the side of the wall extends slightly above the level of the roof, and part of the rain is carried away from the gage. In the center of a large, flat roof, at least 60 feet square, the rainfall collected by a gage does not differ materially from what is collected at the level of the ground. A gage on a plain with a fence 3 feet high around it at a distance of 3 feet may collect 6 per cent more rain than without the fence. The gage should therefore, if possible, be set in an open lot,

A. PRECIPITATION AND EVAPORATION STATION, MADISON, WIS.

B. CABLE STATION, SUSQUEHANNA RIVER NEAR McCALL FERRY, PA.

unobstructed by large trees, buildings, or fences. Low bushes and fences, or walls that break the force of the wind in the vicinity of the gage are, however, beneficial, if at a distance at least as great as the height of the object. Such a place, in general, affords the best exposure. Gages should be exposed on roofs of buildings only when necessary, and then the roof should be flat, or nearly so.

Snowfall is seldom, if ever, satisfactorily measured, because of the difficulty in collecting in a receptacle of any kind the true amount of snow falling in a wind. In order to measure snowfall, therefore, it has been found most satisfactory to arrange a platform on which the snow is allowed to fall, and to collect and melt a vertical sample of known area, thus determining its water equivalent. But even with this method it is difficult to avoid wind effects and to procure a true sample of precipitation. Falls of snow are therefore almost always recorded too small.

The amount of water collected on the small surface presented by a rain gage is assumed to represent the precipitation over a large area. Since, as explained above, slight differences may occur in the amount of water collected, and consequently, in the recorded rainfall, any data in regard to the precipitation over an area may be in error. Moreover, observation stations are generally located in the lower, inhabited sections, and even if accurate records are obtained at the gage, they may not correctly represent the precipitation on the more elevated portions of the basin. It follows, therefore, that the application of a few records to a large area may result in considerable error.

The United States Weather Bureau has collected precipitation records for many stations. The plan followed contemplates the establishment of a precipitation station in each county; therefore, in most of the areas considered the horizontal distribution of the stations is fairly uniform and representative. This is not generally the case, however, with the vertical distribution of the stations, as the location of the gage depends in large measure on the accessibility of a reliable observer and a telegraph station for use in reporting excessive rains. Neither the observer nor the telegraph is generally available at high altitudes. Most of the stations have therefore been located at low or medium elevations, and little is yet known concerning the effect of elevation on precipitation.

The records of precipitation show great variations from season to season and from place to place, with no ascertainable sequence or order. They show also great variations between different sections of the country and for different altitudes and exposures in the same sections. The

mean yearly and seasonal rainfall for any locality is, however, fairly constant and has been determined for many observation stations from records extending over a series of years.

The average precipitation and the range of departure from the average has been determined with reasonable accuracy for many localities in the United States. In Pl. II are shown lines of equal rainfall which have been drawn from the means of records collected in several years at many observation stations. The departures from the mean conditions can be determined for any place only by study of detailed record of precipitation.

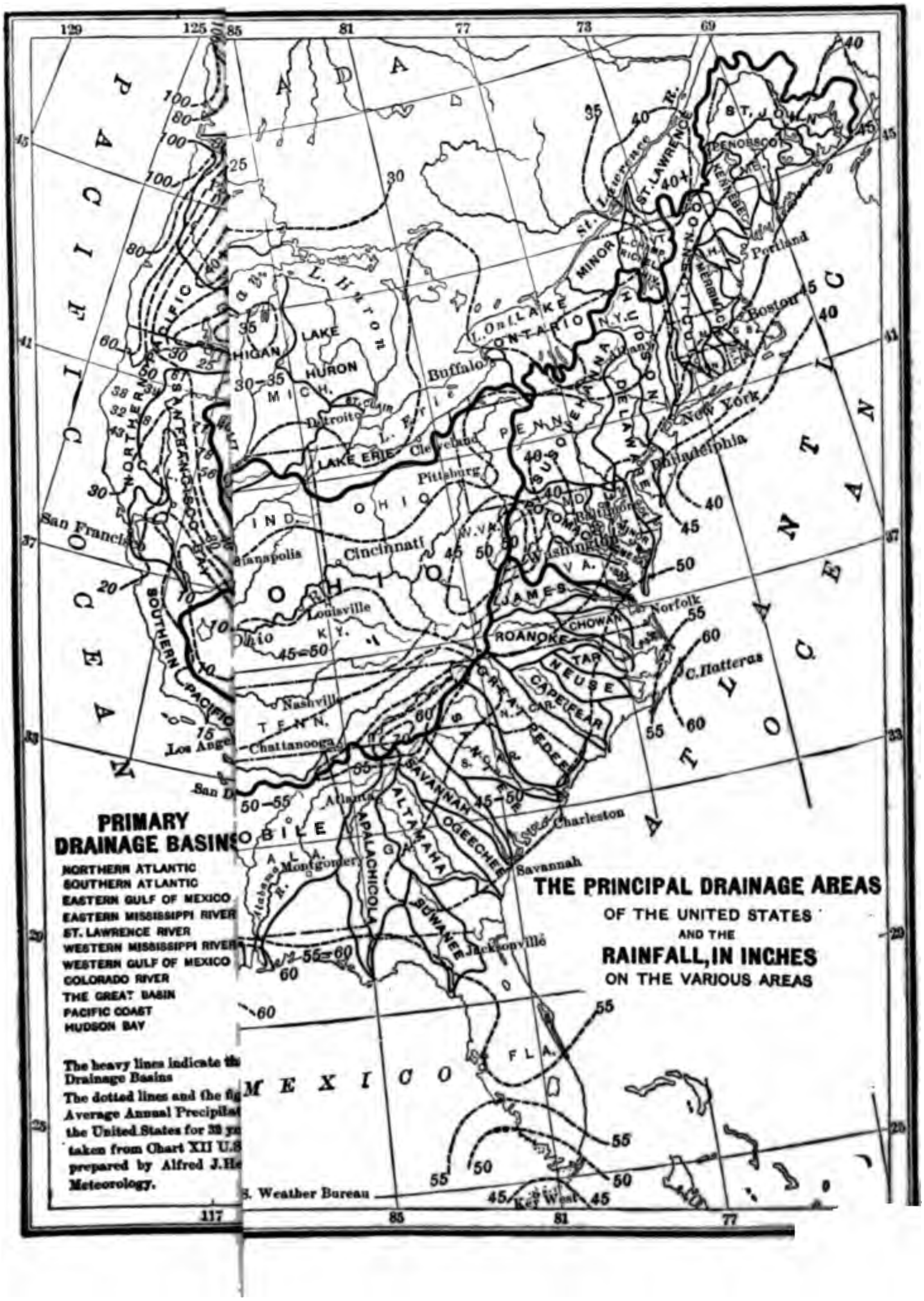
Although a discussion of the conditions that affect precipitation will not be attempted here, it seems desirable to call attention to the effect of variations in latitude, altitude, and topography on the amount and distribution of precipitation, in order that the user of such records may be able to estimate the probable errors in the application of available records to an extended area.

The effect of latitude and altitude on precipitation are similar, since they are the principal factors governing temperature, which affects both the amount of moisture received by the atmosphere and its precipitation. Other conditions being equal, precipitation decreases toward the poles, probably on account of lesser evaporation with decrease of temperature and consequent smaller absolute amounts of moisture in the air. It increases with altitude to the usual height of rain clouds, above which it probably decreases.

The evaporation from water surfaces is in many places so great that the air above such surfaces generally contains a large amount of moisture. If this moisture-laden air is carried by prevailing winds over adjacent land and especially if the lands contain mountains high enough to lift the air currents to those altitudes in which the temperature is sufficiently low to cause precipitation, there will be heavy precipitation on the mountains and between them and the water surfaces. By this process the winds are so robbed of moisture that the lands beyond the mountains receive much smaller amounts of precipitation. On the other hand, if the prevailing winds are away from the land or if no mountains are present to deflect the air currents upward, the adjacent land may be arid.

A study of the annual and seasonal variations of rainfall is of great importance, and a comparison of such rainfall with the measured run-off is necessary in hydrographic investigations. Such comparisons show that great errors will inevitably result from attempts to estimate run-off from precipitation data, and the estimates made

PLATE II.


THE PRINCIPAL DRAINAGE AREAS
OF THE UNITED STATES
AND THE
RAINFALL, IN INCHES
ON THE VARIOUS AREAS
PRIMARY
DRAINAGE BASINS
NORTHERN ATLANTIC
SOUTHERN ATLANTIC
EASTERN GULF OF MEXICO
EASTERN MISSISSIPPI RIVER
ST. LAWRENCE RIVER
WESTERN MISSISSIPPI RIVER
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PACIFIC COAST
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The heavy lines indicate th
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Average Annual Precipitat
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taken from Chart XII U.S
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S. Weather Bureau
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ATLANTIC OCEAN
MEXICO
Buffalo
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Cleveland
Pittsburg
Cincinnati
Louisville
Nashville
Chattanooga
Atlanta
Montgomery
Savannah
Charleston
Jacksonville
Key West
New York
Philadelphia
Boston
Portland
Norfolk
Washington
San Francisco
C. Hatteras

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A study of the actual annual and seasonal variations of rainfall is of great importance, and a comparison of such rainfall with the measured stream flow is of interest in hydraulic investigations. Such comparisons show that great errors will probably result from attempts to estimate stream flow from precipitation data, and the estimates made

PLATE II.


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CANADA

should never be used as the only basis for computations of the amount of water available for power or irrigation.

In using rainfall data the conditions affecting the accuracy of the records, previously noted, must be considered, as well as the possible sources of error stated below.

The record obtained by a single rain gage shows only the measured precipitation on a few square inches of surface. This record, even if accurately made, may not be representative of a considerable area. In order to ascertain with certainty the average precipitation over a large area many rain gages should be employed. Under ordinary conditions of practice, however, the gages in any drainage basin are generally few in number, and as a result the extremes of precipitation, which always occur in comparatively small areas, may not be recorded.

No means have yet been devised for obtaining reliable data in regard to precipitation during the winter months, and practically no data have been collected to show the amount of snow storage at the end of each month. Hence the records of snowfall are especially unreliable, while the lack of information in regard to the amount of water held in the form of snow on the ground makes it necessary to study the winter conditions as a whole.

In any use of rainfall data it is necessary to assume that for any period of time the mean rainfall over the whole of an area is either the arithmetical or weighted mean of the rainfall during that period as observed at the various stations in that area. Since all rainfall records are liable to great errors the weighting of the data is not generally warranted.

In order to compare rainfall and run-off both records should be expressed in "depth in inches" over the drainage basins considered and should of course pertain to the same periods of time. Such data have usually been computed and recorded for calendar months. This period is, however, too short for comparative purposes and may lead to apparently erroneous results, because heavy precipitation which occurs at the end of the month will not appear as run-off until the following month. A year is a better period but is not entirely satisfactory. The calendar year is undesirable as a comparative period, because the conditions of snow and ground storage are not the same at the end of every December. The year beginning with October is a much better period, since on the first of that month the storage conditions are more nearly uniform from year to year, because there is at that time no snow storage, and surface and ground storage are usually at a minimum. The largest disturbing factors in the relation between run-off and rainfall are

Monthly and yearly maximum, minimum, and mean ***loss, for an***

[Note:—M = Mean;

PRECIPITATION, IN INCHES.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford; 3 300 sq. miles....	2.99	4.12 2.12	2.17	5.54 1.05	2.80	4.77 1.36	2.28	2.94 1.99	1.60	3.18 0.71	3.44	4.77 2.05
Housatonic, above Gaylordsville; 1020 sq. miles	3.98	6.49 2.74	2.39	4.29 0.89	4.38	6.76 2.72	3.05	4.89 1.65	2.56	4.24 0.76	4.10	5.20 3.04
Susquehanna, above Harrisburg; 24 030 sq. miles.	3.02	5.74 0.95	2.63	4.43 0.92	2.97	5.63 1.04	2.73	4.40 1.77	2.67	4.55 0.93	3.35	4.58 1.21
Susquehanna, above Wilkes-Barre; 9 810 sq. miles..........	3.33	6.04 1.69	2.41	4.70 1.13	3.30	5.58 2.24	2.57	3.40 1.69	2.30	3.46 1.17	3.60	4.77 3.17
Susquehanna, above Williamsport; 5 640 sq. miles	2.90	6.22 0.89	2.74	4.91 0.51	3.15	5.48 1.25	2.63	3.69 1.51	2.72	4.00 1.05	4.09	5.20 3.42
Ohio, above Wheeling; 23 820 sq. miles.........	2.66	6.78 0.51	3.09	5.67 0.65	3.13	5.07 1.84	3.24	4.97 1.72	3.19	6.54 1.19	3.39	5.62 1.40
Potomac, above Point of Rocks; 9 650 sq. miles..	2.21	6.41 0.57	2.29	4.1[illegible] 0.79	2.67	5.71 0.74	2.53	3.78 1.55	2.91	5.88 0.46	3.42	4.44 2.08
Shenandoah, above Millville; 3 000 sq. miles....	2.47	7.10 0.48	2.08	4.16 0.81	2.61	6.12 0.23	2.58	4.08 1.41	3.12	6.28 0.33	3.52	5.12 2.08
James, above Cartersville; 6 230 sq. miles..........	3.46	8.35 0.55	1.89	3.57 0.93	3.28	7.19 1.41	3.20	4.43 2.21	3.52	5.08 0.59	3.88	6.38 2.59
James, above Buchanan; 2 060 sq. miles..........	2.52	8.07 0.46	2.42	5.20 0.71	3.00	7.63 0.30	2.79	4.56 1.77	3.66	5.80 0.63	3.88	5.66 2.39
James, above Glasgow; 830 sq. miles...........	2.61	8.70 0.32	2.29	5.27 0.73	2.87	7.60 0.17	2.86	4.49 1.80	3.57	5.48 0.49	3.89	6.36 2.67
Appomattox, above Mattoax; 745 sq. miles......	2.70	7.69 0.35	2.20	3.99 1.06	3.76	8.08 1.88	3.28	4.82 2.00	3.11	5.22 0.94	3.63	5.96 2.37
Roanoke, above Roanoke; 390 sq. miles...........	2.71	6.32 0.13	2.49	4.60 1.04	2.92	7.45 0.50	2.85	4.08 1.64	3.92	7.10 0.62	4.01	6.49 2.44
Roanoke, above Randolph; 3 080 sq. miles...	2.61	5.54 0.65	2.22	3.15 1.22	3.95	7.84 1.78	3.10	4.35 2.19	3.56	5.63 0.88	3.57	6.49 2.29

RUN-OFF, IN INCHES.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford..................	1.24	1.94 0.45	1.23	2.51 0.50	1.28	2.34 0.47	0.76	1.11 0.27	0.54	1.03 0.26	3.91	7.56 1.60
Housatonic, above Gaylordsville	1.89	3.25 (0.40)	1.39	1.92 0.96	2.68	4.13 (1.03)	2.32	3.21 0.98	1.61	(3.68) 0.49	5.88	8.77 (4.24)
Susquehanna, above Harrisburg	0.90	2.17 0.16	1.08	2.18 0.28	1.75	3.53 0.49	1.94	3.79 0.67	1.98	4.04 0.53	4.48	7.46 2.46
Susquehanna, above Wilkes-Barre..........	1.16	3.22 0.13	0.96	1.47 0.60	2.53	4.91 0.90	2.89	3.45 2.14	2.56	3.92 1.57	5.33	7.84 2.79
Susquehanna, above Williamsport.............	0.85	2.68 0.15	1.14	1.84 0.29	1.75	4.14 0.33	1.96	3.23 1.01	1.96	4.52 0.56	5.59	8.09 2.82
Ohio, above Wheeling....	0.72	3.70 0.12	1.21	2.97 0.24	1.99	3.64 0.53	2.78	4.30 1.28	3.12	7.29 0.78	4.07	6.89 1.37
Potomac, above Point of Rocks..................	0.52	1.63 0.14	0.41	0.99 0.15	1.06	3.06 0.26	1.30	2.49 0.51	2.00	4.60 0.39	2.80	6.50 1.34
Shenandoah, above Millville....................	0.82	2.99 0.20	0.48	1.05 0.20	1.05	3.12 0.29	1.21	2.62 0.47	1.53	3.63 0.46	2.16	5.34 0.86
James, above Cartersville....................	0.89	2.34 0.21	0.75	1.27 0.26	1.51	2.2[illegible] 0.46	1.80	2.76 0.68	2.11	3.74 0.64	3.16	5.29 1.52
James, above Buchanan.	0.60	2.33 0.13	0.71	2.46 0.20	1.30	4.82 0.26	1.36	2.51 0.42	2.34	5.34 0.51	3.35	5.67 1.53
James, above Glasgow...	0.67	2.48 0.22	0.63	1.67 0.24	1.21	4.50 0.31	1.41	2.79 0.63	2.34	3.99 0.43	2.82	4.44 1.28
Appomattox, above Mattoax....................	0.68	1.44 0.27	0.61	1.32 0.33	1.46	2.51 0.58	1.75	2.73 0.61	2.29	3.60 0.45	2.26	4.01 0.78
Roanoke, above Roanoke.	0.85	3.06 0.26	0.78	1.71 0.24	1.29	4.23 0.35	1.43	3.34 0.22	2.34	5.63 0.64	2.88	7.49 0.94
Roanoke, above Randolph..................	1.03	1.82 0.30	0.81	1.08 0.32	1.71	3.61 0.71	1.63	2.29 0.78	1.98	3.49 1.02	2.33	4.13 1.04

rainfall, run-off in percentage of rainfall, and average year.

R = Range.]

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Total.	R.	
2.77	3.54 1.37	2.99	4.78 0.27	3.78	5.27 2.10	4.34	5.06 3.76	3.88	4.59 3.22	3.73	5.75 1.08	36.76	41.80 33.48	5
3.71	6.12 2.25	2.97	6.24 1.12	5.46	10.42 1.86	5.00	7.25 3.72	5.56	7.28 3.45	4.70	6.42 1.94	47.86	51.49 39.77	5
2.76	4.46 1.27	3.96	7.70 1.27	3.98	6.44 2.77	4.11	7.24 2.42	4.16	6.48 1.92	3.04	5.61 1.41	39.38	45.17 31.62	14
2.70	4.67 1.50	2.73	5.39 1.11	4.46	6.38 2.94	5.05	7.86 4.03	4.49	6.51 2.73	2.90	4.32 1.40	39.85	44.13 31.77	6
2.89	4.69 1.83	3.20	5.41 1.74	4.11	6.03 2.94	4.62	7.58 2.77	4.14	6.62 2.26	2.83	4.70 1.05	40.02	44.11 33.04	10
3.28	6.50 1.57	4.04	7.48 2.18	4.32	5.80 2.50	4.55	9.08 2.54	3.74	6.88 1.30	3.07	6.48 1.56	41.71	55.56 33.47	21
2.61	6.05 1.34	3.77	6.47 1.97	4.15	6.57 1.81	4.15	6.63 2.28	3.50	7.00 1.69	2.65	6.09 1.32	36.86	44.81 29.37	10
2.55	6.24 1.16	3.85	5.82 2.28	4.90	7.63 2.09	4.14	6.21 2.17	3.56	7.73 1.41	2.95	7.22 1.01	38.33	48.08 30.47	10
3.07	6.92 1.72	3.75	6.70 1.78	5.13	7.73 3.55	4.06	7.47 2.35	4.50	10.22 1.54	3.24	4.11 1.98	42.98	54.83 30.58	7
2.66	6.52 1.57	4.20	6.31 1.25	4.78	7.57 3.34	4.42	8.48 2.27	3.67	8.71 1.61	3.17	6.20 1.06	41.17	53.31 30.45	10
2.70	7.08 1.19	4.04	6.18 1.34	4.78	8.71 2.72	4.09	6.22 2.18	3.82	7.47 1.46	3.24	6.70 0.73	40.76	51.48 32.48	10
3.09	5.99 1.08	3.96	7.26 1.72	3.99	5.04 3.20	4.13	7.05 1.94	6.24	13.06 2.70	2.89	4.20 2.29	42.98	52.98 30.80	5
2.80	6.50 1.67	4.18	7.46 0.93	4.77	8.14 1.90	4.91	11.64 3.08	3.80	10.72 0.98	3.32	5.16 1.22	42.68	58.30 35.19	9
3.35	6.04 1.48	4.07	6.33 1.92	4.73	5.98 2.84	4.92	5.09 2.68	5.15	11.21 2.40	2.77	3.29 1.86	43.80	53.95 34.00	5

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	L.	R.	M.	R.	M.	R.	M.	R.	M.	R	Total.	R.	
4.70	7.10 3.64	3.10	4.80 1.16	1.69	3.20 1.02	1.09	1.51 0.49	1.09	1.53 0.69	1.03	1.82 0.37	21.66	27.04 16.01	5
4.63	6.43 3.82	2.40	4.50 1.11	2.24	4.28 0.92	1.47	2.49 0.56	1.41	2.19 0.87	1.51	2.25 1.08	29.43	36.94 23.76	5
3.43	4.83 2.34	2.07	4.54 0.61	1.25	3.03 0.50	0.83	3.25 0.34	0.77	1.60 0.24	0.61	1.42 0.17	21.09	28.03 16.34	14
3.17	4.46 2.49	1.13	2.52 0.40	1.07	1.79 0.40	1.01	3.41 0.28	0.66	1.53 0.12	0.72	1.44 0.15	23.19	27.18 15.15	6
3.50	5.45 2.24	1.68	3.15 0.60	1.20	2.44 0.54	1.23	4.11 0.36	0.87	1.44 0.27	0.52	1.24 0.18	22.26	27.60 16.57	10
3.20	6.84 1.80	1.94	5.10 0.51	1.30	3.37 0.31	1.06	3.49 0.28	0.76	1.88 0.16	0.53	2.50 0.15	22.68	34.20 16.29	21
1.98	4.60 0.76	1.34	3.22 (0.31)	0.99	2.18 0.37	0.76	1.52 0.29	0.69	2.66 0.23	0.34	0.88 0.16	14.22	21.46 8.16	10
1.77	4.79 0.72	1.39	3.36 0.53	1.16	3.07 0.52	0.83	1.71 0.34	0.81	3.15 0.38	0.43	0.93 0.23	13.64	19.78 7.86	10
2.18	4.45 1.01	1.63	3.45 0.94	1.50	3.05 0.68	0.99	1.45 0.38	1.02	3.08 0.30	0.67	1.25 0.29	18.21	24.73 10.69	7
2.02	4.98 0.88	1.77	3.55 0.58	1.17	3.15 0.49	0.98	3.02 0.24	0.81	2.71 0.22	0.50	0.99 0.21	16.91	26.30 11.45	10
1.79	4.20 0.80	1.51	2.83 0.58	1.15	3.00 0.30	0.99	2.82 0.24	0.84	2.49 0.25	0.63	2.14 0.17	15.99	21.33 12.15	10
2.13	3.67 0.85	1.44	3.19 0.92	0.90	1.51 0.48	0.73	1.31 0.39	1.42	4.01 0.53	0.83	1.18 0.37	16.48	25.15 10.92	5
1.89	4.90 0.58	1.79	4.36 0.76	1.14	2.54 0.51	1.16	3.54 0.39	1.33	5.73 0.26	0.80	1.53 0.22	17.69	29.66 8.88	9
1.88	3.49 0.81	1.65	3.16 1.10	1.37	1.74 1.05	1.45	2.43 0.79	1.80	4.94 0.82	1.02	1.45 0.65	18.66	25.16 10.99	5

Monthly and yearly maximum, minimum, and mean loss, for an

[Note: —M = Mean;

RUN-OFF IN PERCENTAGE OF RAINFALL.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford	41	92 17	57	132 33	46	103 17	33	55 13	34	86 23	114	163 45
Housatonic, above Gaylordsville	47	64 (13)	58	175 22	61	67 (37)	76	149 (41)	63	(87) (59)	143	203 87
Susquehanna, above Harrisburg	30	44 6	41	105 11	59	10[illegible] 19	71	133 32	74	209 41	134	277 60
Susquehanna, above Wilkes-Barre	35	53 4	40	85 22	77	140 40	112	201 76	111	197 (47)	148	223 78
Susquehanna, above Williamsport	29	64 10	42	68 12	56	102 15	75	116 50	72	146 43	137	200 74
Ohio, above Wheeling	27	72 6	39	114 12	64	99 27	86	129 46	98	151 54	120	191 82
Potomac, above Point of Rocks	24	162 8	19	41 6	40	76 10	51	98 30	69	168 33	82	149 41
Shenandoah, above Millville	33	623 10	23	64 12	40	383 9	47	78 (27)	49	129 (26)	61	148 23
James, above Cartersville	25	131 13	36	69 23	46	70 21	56	74 31	60	108 34	81	128 44
James, above Buchanan	24	92 8	29	75 14	43	67 11	49	75 24	64	105 16	86	172 40
James, above Glasgow	26	381 8	28	64 11	42	653 12	49	73 34	66	168 51	70	127 37
Appomattox, above Mattoax	23	160 11	29	47 13	39	71 17	53	77 30	74	88 48	62	76 26
Roanoke, above Roanoke	31	208 9	31	77 15	39	92 7	50	90 13	60	152 15	72	115 27
Roanoke, above Randolph	40	180 25	36	89 17	43	53 24	53	71 36	56	116 (31)	65	103 41

LOSS, IN INCHES.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford	1.75	2.60 0.18	0.94	3.03 −0.41	1.52	2.43 −0.06	1.52	1.97 0.91	1.06	2.15 0.10	−0.47	1.92 2.91
Housatonic, above Gaylordsville	2.09	3.24 0.98	1.00	3.33 −0.67	1.70	2.63 0.96	0.73	1.99 −1.05	0.95	2.36 (0.20)	−1.78	0.66 −4.46
Susquehanna, above Harrisburg	2.12	4.16 0.66	1.55	3.42 −0.11	1.22	2.76 0.00	0.79	1.84 −0.62	0.69	2.14 −1.21	−1.13	1.65 −3.58
Susquehanna, above Wilkes-Barre	2.17	3.45 1.27	1.45	3.46 0.17	0.77	1.70 −0.93	−0.32	0.83 −1.72	−0.26	0.65 −1.93	−1.73	0.80 −4.83
Susquehanna, above Williamsport	2.05	4.71 0.46	1.60	3.58 0.21	1.40	2.82 −0.03	0.67	1.50 −0.39	0.76	1.92 −0.84	−1.50	1.02 −4.05
Ohio, above Wheeling	1.94	3.96 0.39	1.88	4.76 −0.37	1.14	2.25 0.03	0.46	1.91 −0.88	0.07	0.90 −1.28	−0.68	0.69 −3.13
Potomac, above Point of Rocks	1.68	4.78 0.57	1.85	3.86 0.64	1.61	2.65 0.18	1.23	2.09 0.05	0.91	2.32 −0.75	0.62	2.60 −2.15
Shenandoah, above Millville	1.65	4.91 −2.51	1.60	3.48 0.29	1.56	3.00 −0.65	1.37	2.11 0.55	1.59	(3.67) −0.13	1.36	(2.40) −1.78
James, above Cartersville	2.57	(6.01) −0.17	1.14	2.67 0.29	1.77	3.90 0.95	1.40	1.93 0.83	1.41	2.31 −0.05	0.72	1.96 −0.92
James, above Buchanan	1.92	2.85 −0.20	1.71	3.83 0.50	1.70	3.07 −0.71	1.43	2.05 0.72	1.32	2.66 −0.03	0.53	2.62 −2.27
James, above Glasgow	1.94	6.22 −0.90	1.66	3.60 0.42	1.67	3.10 −0.94	1.45	1.88 0.86	1.23	2.30 −1.34	1.07	2.55 −0.86
Appomattox, above Mattoax	2.07	6.16 −0.21	1.56	2.67 0.56	2.30	5.77 0.86	1.52	2.09 0.76	0.82	1.62 0.40	1.38	2.27 0.71
Roanoke, above Roanoke	1.86	3.96 0.14	1.71	3.91 0.43	1.63	3.33 0.04	1.42	2.61 0.39	1.57	4.10 −0.32	1.13	2.53 1.00
Roanoke, above Randolph	1.58	3.72 −0.56	1.41	2.13 0.14	2.24	4.23 1.05	1.47	2.06 0.82	1.58	2.73 −0.14	1.24	2.36 −0.09

rainfall, run-off in percentage of rainfall, and average year.

R = Range.]

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Mean.	R	
170	206 120	104	430 65	45	68 20	25	38 13	28	37 16	28	34 22	59	65 46	5
125	187 105	81	118 72	41	113 20	29	61 11	25	40 18	32	80 17	62	73 53	5
124	324 25	52	72 21	31	86 10	20	47 9	18	47 6	20	73 5	55	63 44	14
117	200 96	41	50 26	24	58 10	20	43 7	15	24 4	25	74 11	58	65 47	6
121	276 85	52	69 34	29	66 12	27	54 12	21	82 7	18	56 7	56	63 42	10
98	156 58	48	82 18	30	67 10	23	53 9	20	56 5	17	39 6	54	63 44	21
76	104 35	36	57 (11)	24	48 11	18	34 9	20	38 10	13	28 4	39	61 22	10
69	113 31	36	65 14	24	41 13	20	43 11	23	41 10	15	41 5	36	58 21	10
71	101 48	43	66 32	29	54 18	24	37 12	23	30 11	21	44 8	42	50 33	7
76	123 40	42	79 19	25	52 14	22	38 8	22	39 5	16	80 8	41	58 28	10
66	128 37	37	69 18	24	52 9	24	33 10	22	33 6	19	32 6	39	53 33	10
69	113 42	36	54 24	23	41 13	18	32 9	23	31 14	20	49 14	38	48 30	5
66	89 25	43	125 75	24	57 14	24	48 12	35	53 8	24	37 7	41	56 25	9
56	87 31	41	66 23	30	38 21	29	46 16	35	44 24	37	44 25	43	57 32	5

April.		May.		June.		July.		August.		September.		Year.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R	M.	R.	Total.	R.	
-1.94	-0.70 -3.62	-0.11	1.67 -1.39	2.09	4.23 1.05	3.25	3.81 2.41	2.79	3.62 2.00	2.70	3.74 0.71	15.10	18.81 12.84	5
-0.92	-0.20 -2.20	0.57	1.74 -0.22	3.22	6.14 -0.25	3.53	5.06 1.56	4.15	5.71 2.08	3.19	5.19 0.39	18.43	22.56 13.39	5
-0.67	1.37 -2.84	1.89	3.16 0.58	2.73	5.57 0.40	3.28	4.41 1.94	3.39	5.25 1.22	2.43	4.81 0.53	18.29	21.04 13.54	14
-0.46	0.21 -1.50	1.60	2.87 0.71	3.39	5.30 1.32	4.04	4.53 3.47	3.83	5.10 2.53	2.18	3.76 0.43	16.66	18.61 14.53	6
-0.61	0.47 -2.54	1.52	2.26 0.94	2.91	5.05 1.25	3.39	4.58 2.41	3.27	5.18 0.96	2.31	4.39 0.87	17.76	20.89 15.70	10
0.07	3.00 -1.56	2.10	3.14 0.95	3.02	4.91 1.33	3.50	6.07 2.26	2.98	5.44 1.09	2.53	4.50 0.81	19.02	24.86 15.18	21
0.63	1.60 -0.14	2.43	3.77 1.26	3.16	4.49 1.33	3.39	5.41 1.98	2.81	5.23 1.27	2.31	5.84 1.08	22.64	29.09 13.87	10
0.78	1.83 -0.30	2.46	3.97 1.46	3.74	5.14 1.49	3.31	5.11 1.73	2.75	4.58 1.03	2.52	6.83 0.76	24.69	33.05 14.58	10
0.89	2.47 -0.02	2.12	3.59 0.60	3.63	5.87 2.57	3.07	4.98 1.94	3.18	7.14 1.24	2.57	3.54 1.83	21.77	30.79 18.90	7
0.64	1.54 -0.37	2.43	4.10 0.26	3.60	5.77 2.79	3.44	5.46 1.87	2.86	6.00 1.30	2.67	5.21 0.83	24.26	32.38 14.89	10
0.91	2.38 -0.45	2.53	3.47 0.41	3.63	6.87 1.92	3.10	4.18 1.91	2.97	4.98 1.15	2.61	4.56 0.50	24.77	30.46 16.29	10
0.95	2.51 -0.41	2.53	4.07 0.79	3.09	4.04 2.19	3.40	6.42 1.51	4.82	9.05 2.17	2.07	3.46 1.22	26.50	36.10 19.88	5
0.91	1.70 0.20	2.39	4.91 -0.23	3.63	6.85 0.81	3.75	8.10 2.41	2.47	4.99 0.72	2.53	4.81 0.84	24.99	31.71 15.91	9
1.47	2.55 0.24	2.42	4.95 0.65	3.16	4.75 1.78	3.47	6.30 1.87	3.35	6.27 1.58	1.75	1.99 1.21	25.14	29.38 16.00	5

ground, surface, and snow storage. As data in regard to them are not available as a rule, it has been impossible to allow in any way for their effects.

The table[a] (pp. 10–13) shows, for various drainage areas in the northeastern United States, the monthly and yearly rainfall, run-off, and loss for each of the years for which run-off records are available. The records of precipitation were in some instances incomplete, and figures for several months in the period considered were missing. In such cases the mean of the records for the months available was taken as the mean for the month in question. It may be, therefore, that for January the mean may be obtained from records collected at 20 stations, while for February the mean of 15 records has been used. Interpolation for supplying missing rainfall data adds nothing to the accuracy of the record and is probably never justifiable by theory or facts.

EVAPORATION.

All precipitated water, in some portion of its course over the earth's surface, is subjected to the effects of evaporation, whereby a portion is returned to the atmosphere again and thus disappears from the surface waters.

The principal conditions on which the amount of evaporation depends are the temperature of the atmosphere and of the surface from which evaporation takes place, the relative humidity of the air, and the wind movement. The greatest of these factors is temperature, although the others are important. The greater the wind movement the lower the relative humidity, and the higher the temperature the greater the evaporation. Consequently the rate of evaporation from land surface and from rivers, lakes, canals, or reservoirs varies widely in different localities, and in the same locality in different seasons.

The determination of the amount of evaporated water received by the air from land surfaces is very difficult, and no satisfactory direct measurements have been made. The best information available in regard to this phenomenon has been obtained by subtracting from the total annual precipitation, expressed in inches of depth on the basin, the total annual run-off expressed in the same unit. The difference represents closely the total loss by evaporation from the land and water surfaces of the basin (see table on pp. 10–13).

The measurement of evaporation from water surfaces offers considerable difficulties, but approximate records have been collected at many

[a] See Proceedings of the American Society of Civil Engineers, Vol. XXXIII, May, 1907, from which Pl. II and the table on pp. 10-13 have been taken.

points. These records are of great value in studies of artificial storage, since the total annual storage is diminished by the annual evaporation from the water surface.

The method adopted for measuring the evaporation from a body of water consists in measuring the loss of water from a pan (Pl. I, A), which is so placed that the contained water has as nearly as possible the same temperature and exposure as that of the water which it is intended to represent.

In order that the proper conditions of temperature and exposure may be most nearly attained the pan is either floated in the water at a place where the wind velocity is the average for the body of water under consideration, or, in occasional cases, is sunk nearly to its top in marshy ground. In general, the condition that the pan should be in a place of average wind velocity must be sacrificed to the practical requirement that the water about the pan when it is floated shall be so still that water will not slop in or out and that accurate observation of its height can be made. The location should therefore be chosen with a view to securing a body of relatively quiet water around the pan. The pan should be floated as deep as possible and the water in the pan maintained as near the top as possible, consistent with the requirement that the wind does not drive water in or out of the pan by direct splashing. It is usually not possible to maintain the height above 1½ inches below the rim. The height of water in the pan may be read by means of an inclined scale of such form that the readings, which must be made to hundredths of an inch of vertical elevation, are magnified by the inclination of the scale. It is better, however, to use a point projecting from the center of the pan, fixed at a uniform height, the water in the pan being restored night and morning to such a height that the point is just submerged. The height of water in the pan is thus maintained at practically the same distance below the rim, and the magnitude of errors of observation is reduced.

For the purpose of ascertaining the amount of evaporation a small cup is provided, the cubical contents of which is exactly equivalent to that of the pan at a depth of one one-hundredth of an inch. Such a cup, of cylindrical form, is 4.13 inches high and 2 inches in diameter for a pan 36 inches square. When an observation is to be made the pan should be level. The cup should then be filled with water. This is emptied into the pan and the operation is repeated until the point is exactly submerged. The number of cupfuls emptied into the pan is the amount of water, in hundredths of an inch, added to the pan. If a rain storm has occurred and the point in the center of the pan has been

submerged, water must be dipped out of the pan and the exact number of cupfuls noted.

Reports of the observations should contain data on wind movement, temperature of water inside and outside the pan, amount of precipitation, and amount of water added to or removed from the pan. The evaporation from water surfaces varies from 20 to 40 inches in the humid Eastern States to 70 to 100 inches in the arid West.

In all sections of the country and under all conditions, practically all of the precipitation is either evaporated or appears ultimately as surface water. The only exceptions are the small amounts which are lost permanently to the ground or which undergo chemical change by vegetable growth or by entering into composition with other minerals. The major part of all ground water becomes a part of the surface water again by flowing from springs, or is transmitted directly to the atmosphere by transpiration from vegetation.

The difference between annual rainfall and run-off thus represents very closely the annual evaporation.

The effects of evaporation extend, therefore, both to the total flow of streams, as indicated above, and to the distribution of flow by the different rates of evaporation for different seasons.

The major part of summer precipitation returns in a relatively short time to the atmosphere, while the winter precipitation remains longer on the surface and practically all runs off either directly as winter or spring flow, or later as summer flow from ground water. A comparison of rainfall and run-off for the summer and winter seasons, as shown in the table on page 17, illustrates forcibly the effect of evaporation on the regimen of the streams and the remarkable constancy of the total yearly evaporation in the sections covered by the data. It also shows the direct variation of the yearly evaporation with the mean annual temperature and the remarkably small losses during the period of low temperatures.

TEMPERATURE.

Temperature is the chief factor affecting evaporation and rainfall and thereby affects both the total run-off and its distribution. Low temperature exerts a further effect, mainly on the distribution of flow, by forming snow and ice and temporarily storing the precipitated moisture.

Storage resulting from low temperature is an important factor in the regimen of streams in high latitudes and in mountainous areas. A

few determinations of the quantity of such storage have been made by collecting cylindrical samples of the snow and ice, extending from the surface of the snow to the ground, and melting each such sample to determine the equivalent depth of water. The relation between depth of snow and equivalent depth of water varies with the temperature at the times of the snowfalls and also, generally much more, with the length of time the snows have remained on the ground, because the weight of snow above and the absorption of rains, if any, or of water from melting snows, serve to compact the snow and thereby to increase the water equivalent for a given depth.

Rainfall, run-off, run-off in percentage of rainfall, and loss, for the winter and the summer months, for the mean year.[a]

Station.	Winter Months, Nov. to Apr., Inclusive.				Summer Months, June, July, August.				Total yearly loss.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
Connecticut, at Orford, N. H.	12.89	11.19	87	1.70	12.00	3.87	32	8.13	15.10
Housatonic, at Gaylordsville, Conn.	17.80	17.12	96	0.68	16.02	5.12	32	10.90	18.43
Susquehanna, at Harrisburg, Pa.	14.48	13.58	94	0.90	12.25	2.85	23	9.40	18.29
Susquehanna, at Wilkes-Barre, Pa.	14.47	16.48	114	—2.01	14.00	2.74	20	11.26	16.66
Susquehanna, at Williamsport, Pa.	15.48	14.76	95	0.72	12.87	3.30	26	9.57	17.76
Ohio, at Wheeling, W. Va.	16.23	15.16	93	1.07	12.61	3.12	25	9.49	19.02
Potomac, at Point of Rocks, Md.	14.14	9.14	65	5.00	11.80	2.44	21	9.36	23.64
Shenandoah, at Millville, W. Va.	14.38	7.72	54	6.66	12.60	2.80	22	9.80	24.69
James, at Cartersville, Va.	16.95	10.76	63	6.19	13.69	3.51	26	10.18	24.77
James, at Buchanan, Va.	15.99	10.37	65	5.62	12.87	2.97	23	9.90	24.26
North (of James) Glasgow, Va.	15.89	9.57	60	6.32	12.69	2.98	23	9.71	24.77
Appomattox, at Mattoax, Va.	16.87	9.89	59	6.98	14.36	3.05	21	11.31	26.50
Roanoke, at Roanoke, Va.	16.50	9.84	60	6.66	13.48	3.63	27	9.85	24.99
Roanoke, at Randolph, Va.	17.53	9.53	54	8.00	14.60	4.62	32	9.98	25.14

[a] For the number of years records see table on pp. 10-13.

GEOLOGY.

Aside from its effect on topography, which results directly from it, geology has an important influence on the regimen of stream flow in two ways, which relate to the nature and depth of the soil and the dip of the strata. It affects both the total run-off and its regimen.

Rain that falls upon a sandy soil enters it almost immediately, with a minimum loss through evaporation, is retained there tempo-

rarily, and readily and gradually flows from it to springs and rivers. Clay and loam receive water much less readily and give it up much less freely. Large areas and depths of sand are therefore among the best natural regulators of stream flow.

Basins of bare rock, on the other hand, shed immediately the major portion of the water that falls upon them. Between these two extremes of rock and sand are all grades of clay, silt, and loam, of varying depths, affecting very decidedly the regimen of the streams that drain them.

The dip of the strata and their porosity are often important on account of their effect on the courses of streams and on the concentration of fall, but also on the amount of water absorbed by the ground to appear elsewhere as springs in the same or another drainage basin, or to be lost permanently to the ground, unless it is brought to the surface again by means of some deep well.

TOPOGRAPHY.

The general topography of the country affects largely both the total amount and the distribution of the run-off. Its influence on the total run-off is derived principally from its effect on precipitation. Mountain ranges frequently cause the precipitation of large quantities of moisture upon them, thereby decreasing the moisture and the clouds in the atmosphere, which they intercept, so that little precipitation occurs beyond the range. By their storage of snow, mountainous areas form the principal reservoirs for the water supply of western streams.

The nature of the slopes of the basin and stream affects principally the distribution of run-off. Steep slopes discharge their water rapidly and as a rule store relatively small amounts of ground water. Flat areas, on the other hand, if pervious, absorb much of the water that falls upon them and yield it up gradually.

Rivers draining steep slopes and having steep beds are therefore "flashy" in character, having great ranges in stage and very small minimum discharges. Rivers that drain flat countries, containing swamp or sand areas, fluctuate less rapidly, having fairly uniform stage and relatively large minimum discharge.

The shape and size of a drainage basin has a decided effect on the streams draining it. The flow from basins which are so large that they cover considerable latitude may be materially affected thereby on account of the difference in the times at which the snows are melted. Rivers flowing southward may discharge the snow water without serious freshet, since the snows melt gradually, beginning in the southern part

and gradually extending toward the north. On the other hand, rivers flowing northward are more liable to freshets from the accumulation of water as the higher temperatures advance north, and to ice-jams and consequent freshets from backwater therefrom, because of the greater thickness and strength of the more northern ice. Rivers flowing east and west are more liable to have the same temperature conditions throughout the whole basin and consequently an accumulation of waters resulting from freshets simultaneously in all tributaries. The shape of the basin, whether long or palmate, affects in a similar way the accumulation of freshet waters.

The shape of the basin and the direction of its axis relative to the direction of motion of prevailing storms is often an important factor in determining the magnitude of freshets. Basins whose axes lie in such direction of motion of storms and whose streams flow in the direction of such motion rather than against it are especially liable to excessive freshets.

The surfaces of lakes and ponds affect the regimen of streams in two ways—first, by decreasing the total annual run-off, on account of the great evaporation from their surfaces, and second, by equalizing flow and making the discharge more regular. They are also important on account of their availability for artificial storage. Swamp areas affect streams in much the same way, especially as regards their regimen.

VEGETATION.

Vegetation affects very largely the regimen of streams. Its influence does not probably extend to rainfall but pertains principally to effects on surface and ground storage and evaporation.

The ground storage is increased on account of the greater receptivity of a soil loosened and opened by roots and a surface covered with fallen leaves and litter. The roots and cover retard the flow of water over the surface and thereby promote the absorption of the water by the soils. The effect of forests on ground storage is therefore a minimum on open, sandy soils, which would readily absorb water under all conditions, and a maximum on heavy, compact soils and clays, which do not take up water easily.

The differences between evergreen and deciduous forests or between large and small trees in their effects on ground storage are probably small. Young trees and bushes, or even grass and weeds, produce approximately the same effects in this respect as virgin forests. In their effects on temperature and resultant evaporation, and especially

on snow storage, the differences in forests are great. In the latter respect, especially, dense evergreen growths have much more marked effects than open or deciduous growths. Unfortunately the magnitudes of their effects have not been measured, and the conclusions in regard to them are based rather on general observations than on measured differences in stream flow. The variety of conditions existing within basins of considerable size makes exact determinations of the effect of the various factors very difficult. Many attempts have been made to study the magnitude of the effects of forests on stream flow, but without satisfactory success. In order that such observations may result in definite conclusions it is necessary that conditions of climate, geology, and topography shall be the same, and that the only differences shall pertain to forestation. Such conditions are not readily found or produced.

ARTIFICIAL CONTROL.

Artificial control of the flow of surface water may be effected either by storage in ponds, lakes, or reservoirs, or by drainage of swamp or wet areas by drains or open ditches.

Artificial storage is generally cheapest and its results greatest in glaciated regions, where natural ponds and lakes available for increased storage are numerous. By means of such artificial storage freshet flows are diminished and low flows increased.

By artificial drainage, on the other hand, the opposite effects are produced. It is believed that a considerable proportion of the effects on stream flow usually ascribed to deforestation and cultivation are due to the artificial surface and subsurface drainage which has been accomplished incidental to cultivation.

CHAPTER III.

INSTRUMENTS AND EQUIPMENT.

The establishment and maintenance of stations for determining the discharge of a river require the use of certain instruments and equipment. These consist of instruments for measuring velocity, gages for determining stage, cables and their appurtenances for providing a support for the engineer in making the measurements, stay lines for holding the meter in position, recording devices, and other appliances.

INSTRUMENTS FOR MEASURING VELOCITY.

Two principal types of instruments are used for measuring the velocity of flowing water—floats, which measure the velocity directly, and current meters, by which the velocity is obtained indirectly from observations of the number of revolutions of the wheel. Another instrument sometimes used for measuring velocity is the Pitot tube, but it is not practicable to use this tube for the work discussed in this book.

FLOATS.

Floats are utilized for the direct measurement of the velocity of streams. Those in common use are surface, subsurface, and tube or rod floats.

Surface floats.—A corked bottle with a flag in the top and a weight in the bottom makes a very satisfactory surface float, as it is but little affected by the wind. In flood measurements good results can be obtained by observing the velocity of débris or of floating cakes of ice. In all surface-float measurements coefficients must be used to reduce observed velocities to the mean velocity.

Subsurface floats.—The subsurface float is designed to measure velocities below the surface and may be made to float at any depth. By arranging the submerged float at the depth of mean velocity it may be utilized in observing mean velocity directly. Allowance must be made, however, for the accelerating effect of the attached line and surface float.

Tube or rod floats.—The tube or rod float is designed also to measure directly the mean velocity in a vertical. It is generally a cylinder of tin, about 2½ inches in diameter, weighted at its lower end and plugged with wood or cork at its top. Small extra weight to make it float at the exact depth desired may readily be added by admitting water or by putting in shot. The tube should be graduated, and alternate feet painted black and red in order that the depth of flotation may be readily observed.

A number of tubes, of different lengths, are necessary for measuring the velocity at different depths in an ordinary cross-section. A float of this type is consequently best adapted for use in artificial channels, in which the depth is nearly uniform, as natural channels are generally too rough and too variable to permit its satisfactory use.

Although designed to measure directly the mean velocity in a vertical, the tube can not be made to float in contact with the bed of the stream, and consequently it does not receive the effect of the slowest moving water. The rougher the bed the greater the error in this respect. A factor less than unity is therefore necessary to reduce the observed velocity to the mean.

CURRENT METERS.

The essential parts of meters for measuring the velocity of water are a wheel of some form, so constructed that the impact of flowing water causes it to revolve, and a device for recording or indicating the number of revolutions of this wheel. The relation between the velocity of moving water and the revolutions of the wheel is determined by rating each meter.

Many kinds of current meters have been constructed, each type having been developed for use under some special condition. Among the meters in most common use are the Price, the Haskell, and the Fteley.

Price meter.—The Price current meter[a] (Pl. III, A) is probably best adapted for general use in making velocity determinations under a wide range of conditions. It is made in two general styles—one with an electric device for indicating the revolutions to the ear, and the other with a direct acoustic attachment. These are usually designated, respectively, as the electric and acoustic Price meters. The essential features of the two meters are the same, the difference being principally in the sounders.

The electric meter (shown at the left in Pl. III, A) consists essentially

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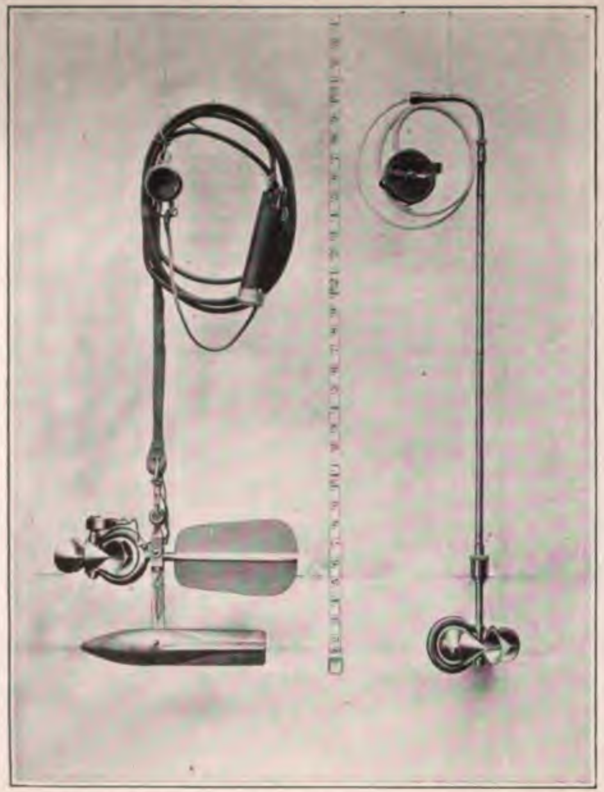
[a] Manufactured and sold by W. & L. E. Gurley, Troy, N. Y.



A. PRICE CURRENT METERS.

B. HASKELL CURRENT METER.

of a ⊂-shaped yoke carrying a wheel made of conical cups set with the axes of the cones tangent to a circle. This wheel revolves in a horizontal plane on an axis that extends vertically between the ends of the ⊂ and turns on a conical point bearing. The yoke and wheel form the head of the meter and are counterbalanced by a two-bladed tail, which is utilized also to keep the meter headed in the current. The meter is hung pivotally in a stem or hanger, provision being made for a slight tilting motion. Its stability in the water is maintained by lead weights. Under ordinary conditions the meter is suspended by a No. 14 double-conductor insulated cable, which also provides for the transmission of the current to the sounder.

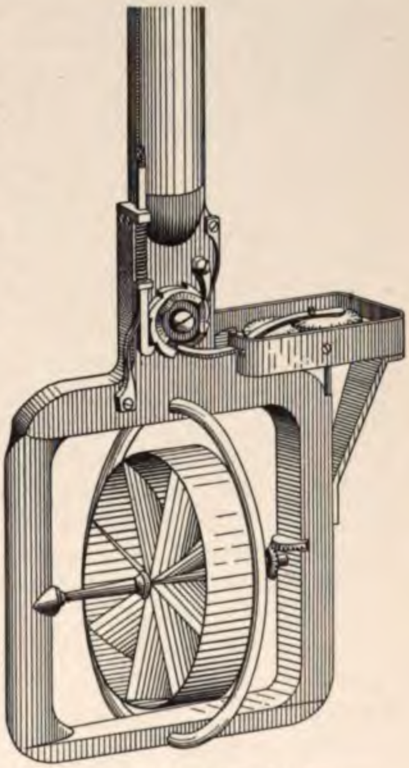


FIG. 2.—Fteley Current Meter.

The sounder, which indicates each revolution of the wheel, consists of either a telephone receiver or an ordinary buzzer excited by either a dry or a wet cell. The suspending cable, carrying the sounder, is attached to the meter by two small insulated wires, which are connected respectively to the stem and the contact spring in the head, the latter through a hard rubber bushing. The upper end of the axis on which the cups revolve carries an eccentric post, which is so arranged as to bear against the contact spring once during each revolution of the cups, thus closing the circuit and exciting the sounder. The head, which carries the shaft and contact spring, is made water-tight, thus insuring that the contact shall be free from disturbance. The electric meter is made in two sizes, known as the large and the small Price electric, the small being intended for measurements of low velocities and the large for high velocities. The general features of the small Price, however, are much better than those of the large Price, and when provided with a reducing device which indicates only every fifth instead of each revolution of the wheel it can be used satisfactorily for measuring high as well as low velocities.

[illegible] The time and [illegible]

[illegible] corresponding [illegible] Determinations are [illegible] cause the wheel to [illegible] of these runs, when [illegible] velocity per second as [illegible] rating-curve, which [illegible] From this curve the velocity [illegible] revolutions can be obtained.

[illegible] Brookline, Mass.
[illegible] Boston, Mass.

Meters are not generally reliable for measurements of velocity under 0.3 foot per second, because a small velocity is required to start them.

It is interesting to note that a meter running at ordinary speed passes through water as a wheel passes over ground; that is, in going a certain distance it will make practically the same number of revolutions regardless of the speed.

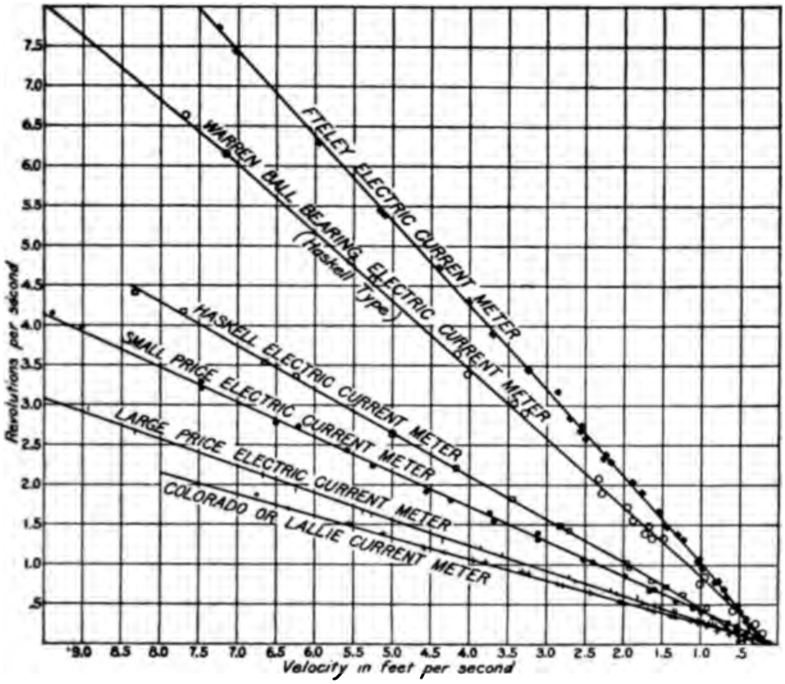


Fig. 3.—Typical Current-Meter Rating Curves.

GAGES.

The gage is the instrument, graduated scale, or other device whereby the stage and changes in stage are observed or recorded. This fluctuation is measured with reference to a fixed datum, to which the position of the gage must maintain a constant relation. The many styles of gage in use all belong to two classes, recording and non-recording.

RECORDING GAGES.

Recording gages make a record of stage either continuously by a curve or at stated intervals by a printing device. The essential parts of the recording gage are (*a*) a float which rises and falls with the

surface of the water; (*b*) a device for transferring this motion of the float to the record, either directly or through reducing mechanism; (*c*) the recording device; and (*d*) the clock. The Frieze gage (figs. 4

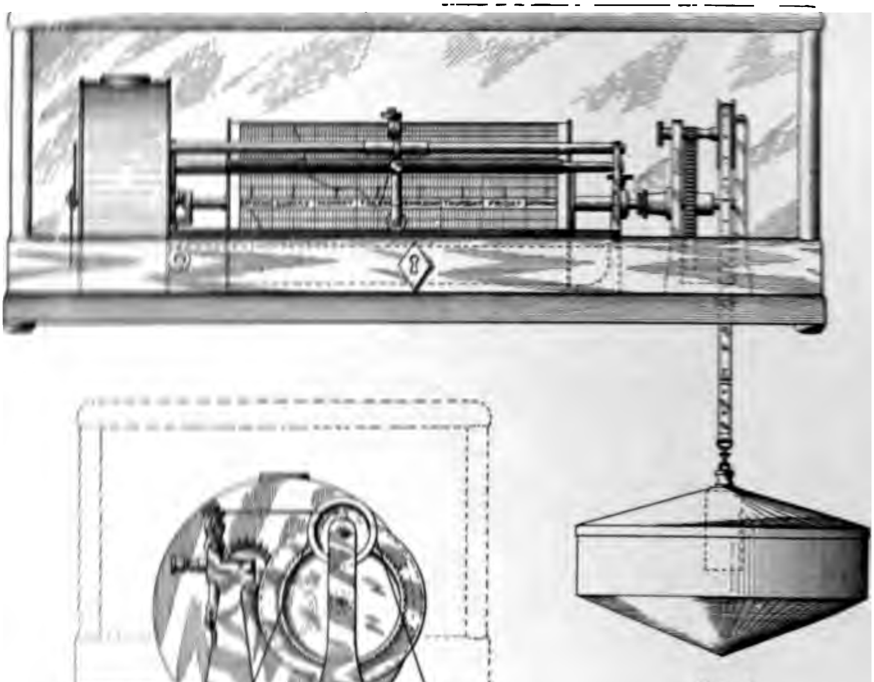


Fig. 4.

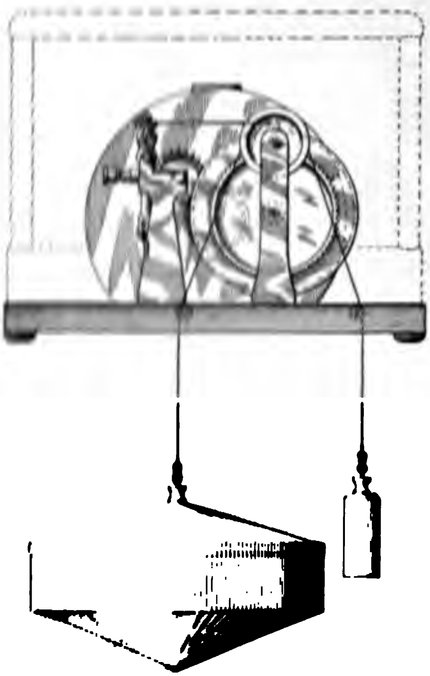


Fig. 5

Fig. 4.—Frieze gage, front view with float and counterweight in position; cover case in place and sample of an actual record for four days (Saturday to Tuesday, both inclusive). Fig. 5.—End view of same, from the right.

and 5) illustrates one of the best types of recording gages which makes a continuous record.

Recording gages are at present generally limited in use to the measure-

ment of small fluctuations in stage. They are especially valuable for recording the stage of canals. In their use the float should operate in a stilling box in order to eliminate wave action, and care must be exercised to maintain a constant relation between the graph and the datum plane.

NON-RECORDING GAGES.

The various forms of non-recording gages may be grouped into (1) fixed graduated staffs or scale boards, on which the water rises directly, and (2) weight gages, in which the elevation of the water surface is obtained by measuring downward from a fixed point. Gages of both classes should be graduated to read directly the elevation above the datum and should be so located that they are easily accessible and beyond the influence of disturbing effects, such as boils, backwater, and cross currents.

Staff gages.—Fixed staff gages may be either vertical or inclined. They have the advantage of certainty in datum so long as the gage is undisturbed, small first cost, and simplicity in reading. They have the disadvantage of being liable to disturbance or destruction by frost action or by floating ice, logs, or drift. The requirements for a satisfactory gage of this class are (1) that the graduation be both clear and permanent; (2) that the gage be easily accessible to read; (3) that it be stable.

The vertical staff is the better of the two when there is available, either in or over the water, an artificial or natural object having a vertical face to which the gage may be attached. Such object may be a bridge abutment or pier, a wharf, a tree, or a rock.

The best form of vertical gage consists of a base of rough 2-inch by 4-inch or 2-inch by 6-inch plank, to which a lighter plank having the graduated face may be easily fitted and nailed, with the zero at the desired elevation. The graduated plank will be found satisfactory if made in about 5-foot sections of ⅞-inch by 4-inch pine, painted white, with graduations cut as V-shaped notches painted black. This facing and graduation is cheaply made and easily installed, and the graduations are reasonably permanent.

The inclined gage is useful where there is no existing object to which a vertical staff may be attached. It should be made of 4-inch by 4-inch timber, supported at short intervals on posts firmly set in the ground, and should be graduated by level after being placed in position so as to give the readings directly. Such gages are especially liable to

change of datum and should be frequently checked in elevation at several points.

Weight gages.—Weight gages are used where a fixed staff gage would be in danger of frequent disturbance or where it can not be so placed as to be easily accessible. The simplest form of this gage consists of a graduated rod or tape, which the observer uses to measure vertically down to the surface of the water from the reference mark on a bridge, vertical ledge, or overhanging tree. The record of stage obtained by this means should be adjusted to read directly from the datum.

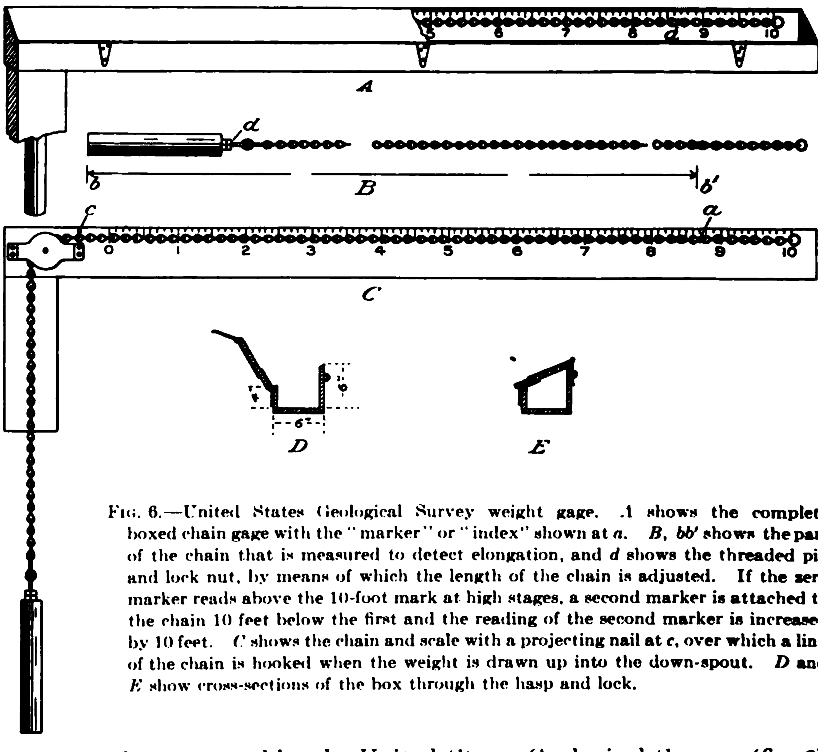


FIG. 6.—United States Geological Survey weight gage. A shows the complete boxed chain gage with the "marker" or "index" shown at a. B, bb' shows the part of the chain that is measured to detect elongation, and d shows the threaded pin and lock nut, by means of which the length of the chain is adjusted. If the zero marker reads above the 10-foot mark at high stages, a second marker is attached to the chain 10 feet below the first and the reading of the second marker is increased by 10 feet. C shows the chain and scale with a projecting nail at c, over which a link of the chain is hooked when the weight is drawn up into the down-spout. D and E show cross-sections of the box through the hasp and lock.

The weight gage used by the United States Geological Survey (fig. 6) is the most practical gage of this class. It consists of a graduated scale board, 10 feet or more in length, either extending from or contained in a box supporting a pulley wheel, over which runs a heavy sash chain, to which is attached at one end a weight and, near the other end, a marker. This as a whole is fastened in a horizontal position to a bridge or other structure, so that the weight when lowered will come in contact with the water. These gages should be erected over moving water, as

the exact point of contact of the weight and water can not easily be determined by the observer above if the water is still. The accuracy of the record obtained by a gage of this type depends on keeping constant the length of the chain between the marker and the end of the weight.

Generally the scale board is graduated only for a length of 10 feet. If the range of stage is greater than that amount provision must be made for measuring it. This is accomplished by a second and, in extreme cases, a third marker, spaced at 10-foot intervals.

To read the chain gage the observer releases the chain and allows the weight to lower until it just touches the surface of the water, in which position the stage is read on the graduated scale opposite the marker. This gage has the advantage of stability in position, as it is above all danger from ice and drift. It has the disadvantage of possible uncertainties in the datum, caused by change in length of chain, due to the wearing of the many parts moving upon one another, and by changes in elevation of the structure to which it is attached. To avoid error the chain length—that is, the length from the end of the weight to the marker—must be frequently measured and adjusted to the standard length. This adjustment is made either by cutting out a link or by the screw adjustment with which the chain is attached to the weight.

Several substitutes for the chain in this form of gage have been or are now being used. Woven-wire sash cord and various forms of wire are liable to kink and are not easy to adjust in length. Various kinds of steel and bronze tape have been tried with some success but in general they have been found liable to break and expensive to mend. They offer considerable resistance to the wind, which makes it difficult to take accurate observations when the distance above the water is great and there is much wind.

Fig. 7.—Hook Gage

Hook gages.—The hook gage (fig. 7), invented by Boyden about 1840, is used for the precise measurement of stage. This gage consists of a vertical graduated rod, carrying a hook at the bottom. The rod slides in fixed supports provided with a vernier for reading. The hook is submerged and by means of a tangent screw is gradually raised until the point just pierces the surface of the water so as to show as a pimple resulting from capillary action. By careful adjustment such a gage can be made to read to a thousandth of a foot. This gage must always be set in a stilling box to insure absence

The cable.—Iron or steel cable of sufficient tensile strength to sustain the car and two men, in addition to the weight of the cable itself, should be used. The stress in the cable due to a vertical load will increase as the sag decreases. Consequently the cable is least safe when the sag is a minimum. In the following table the diameter is computed for a live load of 450 pounds on the cable at the center of span and an initial tension corresponding to the sag given in the table. With an ultimate strength of 80,000 pounds per square inch the factor of safety for these dimensions is about 5. The sag given in the table is the least allowable; if it is increased, the factor of safety is increased. In making connections, the cable should not be bent to a shorter radius than three diameters and the turnbuckle and connections should have a safe working strength of an amount given in the last column of the table. Galvanized cable, pulley, etc., should be used, in order to delay corrosion.

Proper diameter and sag of galvanized steel cable, with live load of 450 pounds for spans of 100 to 800 feet.

Span.	Diameter.	Sag.	Stress.
Feet.	*Inches.*	*Feet.*	*Feet.*
100	$\frac{1}{2}$	4	2,938
200	$\frac{9}{16}$	6	4,167
300	$\frac{5}{8}$	8	5,061
400	$\frac{3}{4}$	10	6,300
500	$\frac{3}{4}$	12	7,813
600	$\frac{7}{8}$	12	10,125
700	1	14	12,026
800	$1\frac{1}{8}$	15	16,660

Supports.—The nature of the supports for the cable will depend on the physical characteristics of the location. It may be supported either by some natural object, as a tree or cliff, or by some form of artificial tower.

Frequently trees are properly located to serve as supports, and when so located may be cheaply and satisfactorily used. The only objection to them arises from their swaying in the wind. Protection in the form of wooden blocks must be provided for the limbs which support the cable to insure that the motion of the tree shall not speedily cause the destruction of the support. A better way, when possible, is to pass the cable through a pulley block, which, in turn, is attached to the support. Large rocks, when available at sufficient elevation above the stream bed, make excellent cable supports, as the cable can be connected directly to the anchorage.

In case artificial supports are required the form will depend somewhat on the height necessary. For low support and a short span, a single

post, 10 to 14 inches in diameter, set firmly in the ground, is sufficient. When, however, heights greater than 12 or 15 feet are necessary, "shear legs" (Pl. IV, A) are generally used. In their construction two posts (8 inches by 10 inches or their equivalent in round logs) should be set in the ground 10 to 15 feet apart at the base, inclined toward each other so that they will be 2 to 5 feet apart at the top, and connected by at least three strong pieces secured to them by bolts fitted with washers and nuts or by "drift bolts" of suitable lengths; or these posts may be set so that they will cross near their ends, and should then be fastened to each other by two or more bolts with nuts. The cable may rest on the top cross bar in the first instance or in the crotch in the second instance, but in either case should preferably be passed through a pulley block at the end having the turnbuckle. All towers should be well guyed so they can not move toward the stream. In crossing the shear legs the cable should make equal angles with the legs on both sides.

Anchorage.—The form of anchorage will vary with different conditions. If solid rock is available, an eye-bolt split at the lower end and driven against a wedge may be set in a drill hole, which should then be completely filled with sulphur, lead, or Portland cement grout. If no solid rock is at hand, a "deadman," made of a log 8 to 12 inches in diameter, may be buried in the ground below the limits of frost and at least 4 feet deep, the length of the log and depth in the ground depending somewhat on the span of the cable.

The anchorages should be so arranged by means of long eye-bolts embedded in concrete, or auxiliary cables attached to the "deadman" that the main cable and its connections will be exposed for inspection.

The cable should be attached at each end to two independent anchorages or supports. In case posts are used for supports the cable should be attached to them by means of a short piece of cable with clips. A support which is not set in the ground should be guyed to anchors of some kind, both forward and backward, and the cable attached to it. In still other cases it is advisable to make a second independent anchorage in the ground.

Turnbuckle.—A turnbuckle for use in taking up sag, having a capacity of 2 to 6 feet, should be inserted in the cable on the side of the river from which the engineer approaches the station. This may have right-and-left screws or a screw at one end and a swivel at the other.

An arrangement can easily be made whereby one man alone can tighten the cable, even if a greater length than the capacity of the turnbuckle must be taken up. This is accomplished by means of an auxiliary cable, which spans the turnbuckle and is clipped to both the

main cable and the anchorage. The turnbuckle having been unscrewed and in that condition clipped to the main cable, the auxiliary cable is released and the turnbuckle drawn up. If the capacity of the turnbuckle does not remove a sufficient amount of sag, the auxiliary cable must again be clipped to the main cable and the turnbuckle released, unscrewed, and slipped along the main cable to a new position and the operation repeated.

Car.—The car should be made about 5 feet by 3 feet and about 1 foot deep and attached at each end to a pulley on the cable by means of iron or steel straps or by light cable, or by wooden standards; never by manila or cotton rope. If wooden standards are used, they should be so securely attached to the car that in case of accident they will not be wrenched loose. Pl. IV, B, shows an excellent type of car, which has the advantage of great comfort as compared with a car having a closed bottom.

The pulley wheels which support the car on the cable should be 10 to 12 inches in diameter for a span of 600 feet or more and should never be less than 6 inches in diameter. In case of the longest spans indicated above, it will be found advisable to arrange cranks for propelling one of the wheels, and a clamp should be provided whereby the car may be held firmly in position on the cable. These attachments decrease materially the labor of drawing the car up the incline of a cable. The two pulley wheels must be kept at an equal distance apart by means of a wooden or metal stretcher.

STAY LINES.

If the stream is deep and its velocity is great at high stages a stay line equipped with suitable pulleys should be stretched across it 100 feet or less upstream from the cable and used, when necessary, to hold the meter in position, as shown in Pl. I, B.

WEIGHT.

The shape of the lead weights used for holding the meter in position in the water is of considerable importance. Flat-iron shapes have been generally used and are fairly satisfactory. Torpedo shapes, shown attached to the meters in Pl. III, give the best results. In the swift water of high stages several weights of 10 pounds each must be used to keep the meter submerged. For such use a number of flat weights may be attached by means of a special stem, or special heavy weights

may be made. Extra heavy and large tail pieces, with both horizontal and vertical vanes, must also be attached to the weights in order to keep the meter headed properly into the current.

SOUNDING APPLIANCES.

The most common sounding appliances in general use are rods and weight and line.

Rods are limited in use to depths of less than 15 feet. They should be round in order to be easily handled and may be made either of gas-pipe or of wood. The graduations should be so marked as to be easily read, and in order to avoid sinking into the bed of the stream the bottom of the rod should be protected by a shoe at least 3 inches in diameter.

Weights and lines of many forms are in use and are manipulated either directly by hand or by means of a sounding-reel in case of very deep soundings. The line should be of some material which does not shrink or stretch on wetting. For reels piano wire is generally used. The best form of hand line for use at bridges is a combination of sash cord, which can be easily grasped with the hands, for the upper part, and picture wire, which offers but little resistance to the current, for the lower part.

The shape of the weight should be such as to offer small resistance to the water, and the amount of weight required will depend on the depth and velocity of the current.

STOP-WATCHES.

Stop-watches are necessary for the satisfactory observation of velocity by floats and for integration by meters. They are recommended for use in all meter work.

RECORDERS.

Recorders or indicators to show the number of revolutions of the meter are of two classes, automatic and sounders.

Several types of automatic electric recorders are made by various instrument makers and are useful under certain conditions in stream-gaging work. The sounder, either electric or mechanical, is, however, generally used, because the attention of the engineer is thereby directed to the work of the meter, and any interruptions or irregularities are noted and investigated.

The most satisfactory sounder in use is the telephone receiver, shown attached to the meter in Pl. III, A. It consists of an ordinary telephone receiver and dry-battery cell.

A buzzer excited by either a dry or a wet cell is satisfactory for indicating revolutions of the meter. For such a device a cell charged with bisulphate of mercury and water is convenient, reliable, and inexpensive.

The mechanical sounder used on the Price acoustic meter (Pl. III, A) consists of a drum against which a hammer strikes at each fifth or tenth revolution of the meter.

CHAPTER IV.

VELOCITY-AREA STATIONS.

SELECTION.

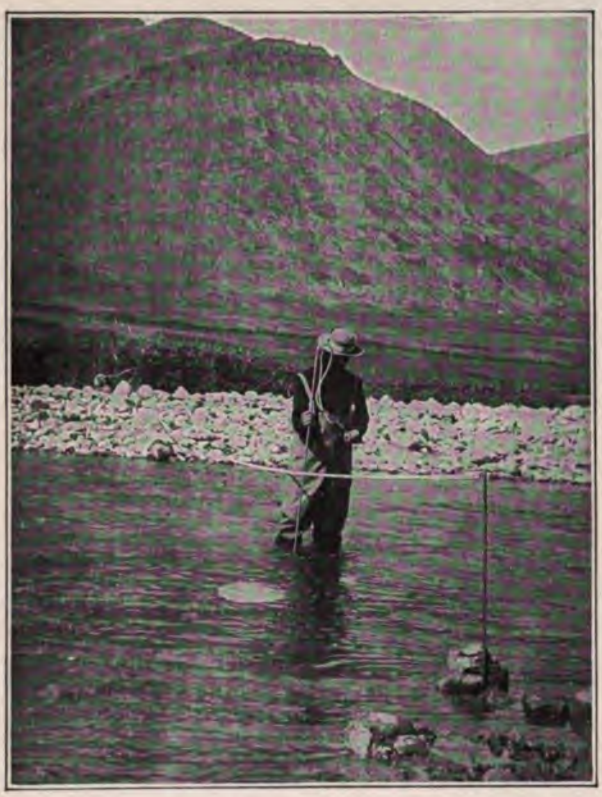
REQUISITE CONDITIONS.

Classes of stations.—Velocity-area gaging stations are classed with respect to the method by which the velocities are measured, into current-meter, float, and slope stations. Current-meter stations are further subdivided with respect to the facilities for making the observations, into bridge, cable, boat, and wading stations (Pl. IV, A; Pl. VII, A; and Pl. V, A). The conditions which determine the desirability of a site for these stations may be divided into three classes. The first class includes conditions which insure good measurements of discharge; the second, those which are necessary for computing the flow at times when measurements of discharge are not made; and the third, those which affect the cost of obtaining the records.

Conditions pertaining to measurements of flow.—If the measurements of discharge are to be made by current meter the requisite conditions are (1) a fairly smooth bed; (2) a measurable and uniform velocity of current; and (3) a stationary stage during the measurement. The velocity of the current should be uniformly distributed throughout the section, which should show no marked eddies, cross currents, or boils, and its mean should not be less than 0.5 foot per second at low stages.

If the measurements are to be made by floats or by slope determinations there must also be a straight stretch of channel, from 200 to 1,000 feet in length, through which the cross-section and velocity are reasonably uniform.

Conditions pertaining to computation of flow.—In order to compute the flow at times when discharge measurements are not made it is essential that the relations of stage to discharge shall be practically permanent. This condition does not exist within the backwater above a dam at which water is used, nor within the backwater from a tributary stream; nor, in general, in localities where the conditions near the



A. TYPICAL WADING STATION.

B. CURRENT-METER RATING STATION AT LOS ANGELES, CAL.

gage change so rapidly that the ratio of gage height to discharge varies from day to day.

Attention is particularly called to the fact that permanence of flow past the gage is the essential condition, because the records of gage heights and the rating table pertain to the section of the gage and not necessarily to the section in which discharge measurements are made. This generally involves the requirement that the bed and banks shall be permanent not only at the gage, but above and below it, because the shifting of a bar of sand or gravel in the channel below will frequently make a decided change in the relation between discharge and gage height, even though the cross-section at the point of the measurements remains unchanged. On the other hand, a permanent reef or ledge extending across the stream a short distance below the gage will control the relation between gage height and discharge, even though the bed at the measuring section itself may change. Stations located under such conditions of control have given excellent records.

If the conditions of control are permanent, measurements made during different years will define a curve which may be applied to a record of gage height extending over an indefinite time to obtain estimates of discharge; whereas if conditions are unstable a rating curve can be developed only for a period during which the conditions remain unchanged.

In some sections of the country it is impossible to obtain the permanent conditions outlined above. If a station is established, however, at which the relation of gage height to discharge is not practically constant, it may be necessary, in extreme cases, to make several measurements of discharge each week.

Conditions pertaining to cost of records.—The cost of obtaining records depends on (1) the accessibility of the station; (2) the availability of the gage reader; (3) the availability of structures or equipment by means of which measurements of velocity and depth may be made, and, in absence of a suitably located bridge, conditions of bank favorable to anchorages and supports for a cable; and (4) the permanency of the bed, which determines the number of discharge measurements necessary for computing the regimen.

In general it has been found economical, in the end, to locate a gaging station where conditions are permanent, even though the cost of each discharge measurement may be much greater than it would be if the station were located at a point more readily accessible but with changing conditions.

RECONNAISSANCE.

A gaging station should be established only after a thorough reconnaissance has been made of that portion of the river in which the station is to be located. As the object of this reconnaissance is to find the best location to furnish the desired results, it should be made, if possible, during a low stage of the river and should be supplemented by an inspection at a high stage, if feasible. At low stages the bed and the minimum velocity can be carefully examined and a fair estimate of high-water conditions can also be made. At medium and high stages it is generally impossible to examine the bed or to make any estimate of velocity at low stages.

In making the reconnaissance, consideration should be given to the type of station to be established, the economic value and the use to be made of the results, and the funds available.

Careful notes and sketches should be made covering the conditions at the several localities examined. These notes are necessary because the final location of the station can not be chosen until all possible sections of the stream have been inspected. They should be complete in all details and should include negative as well as positive information. They should include information in regard to—

1. The accessibility of the site.
2. The type, dimensions, and location of the gage or gages.
3. The availability of gage readers and their qualifications.
4. The estimated cost of establishment of the station.
5. The estimated annual cost of maintenance.
6. The structures available for supporting the engineer in making measurements, or, in the absence of such structures, the span, supports, and anchorages necessary for a cable, with a statement whether or not all flood water passes under the structure or cable.
7. The velocity and distribution of the current of water.
8. The proximity of dams or tributaries above or below the section and their probable effects at the station.
9. The bed—whether rough or smooth, permanent or shifting.
10. The banks—shifting or permanent, wooded or clear, high or low, etc.
11. The section of river available for slope determinations when measurements of discharge are to be made by the slope method. These should include estimated length, curvature, slope, obstructions, facilities for measurement of cross-sections, etc.

ESTABLISHMENT.

The selection of a site for a gaging station must be determined largely by the facility afforded by the several localities available for obtaining an accurate record of stage and for measuring exactly the area of cross-section and the velocity of the current. The routine of establishment is somewhat different for stations of different classes and is described under the headings below.

BRIDGE STATIONS.

Establishment of gage.—Before the gage is installed the gage reader should be selected. The gage or gages should then be conveniently located for reading and the datum or zero placed well below the lowest water. In order to accomplish this it is generally advisable to put the zero at the approximate elevation of the bed of the river at the lowest point in the section. If the gage is not in the section in which meter measurements are made an auxiliary gage must be placed in the measuring section, in order that a standard cross-section may be used for determining areas as factors of the discharge.

If a weight gage is erected, the length of the chain from the end of the weight to the marker should be measured carefully under about a 12-pound pull. In order that this measurement may be made easily the marker should be placed a few feet from the end of the chain. Nails properly spaced in the floor of the bridge will facilitate this measurement and will be serviceable in future checkings of the chain length, which should be made at each subsequent visit of the engineer to the station. The engineer should paint or mark plainly on the inside of the cover of the gage box the length of the chain and the elevation of the reference point from which stage can be determined.

The datum of the gage should be noted with reference to at least two bench marks, one being permanent and preferably placed on an object apart from the structure to which the gage is attached, out of reach of possible damage or interference, and the other fixed on an easily accessible part of the bridge or on an overhanging tree or rock, from which the stage of the river may be directly determined by measurements made from it to the surface of the water by a staff or steel tape. This latter point, generally known as the reference point, should be as permanent as possible and not generally any part of the gage or gage box.

The elevation of the bench marks should always be determined and expressed above the datum of the gage without reference to an intermediate datum.

In order that the gage heights may be readily used in flood studies and in determining slopes along the river, the datum of the gage should be, whenever possible, connected with sea level or with any city or railroad datum available. In making the original reference and in future comparisons of the gage with its bench marks the level should first be so set as to obtain directly the height of the instrument above the datum of the gage. In the case of a staff this can be accomplished by reading directly from the gage, or by setting the bottom of the level-rod at some definite point on the gage. For the standard weight gage the instrument should be set below the elevation of the pulley and the gage weight lowered until its bottom is on a level with the horizontal cross-hair. The reading of the gage in this position gives directly the height of the instrument.

The height of the instrument should not be measured from a water level, because the elevation of the surface of the river may vary materially within its width or within short distances up and down the stream.

Initial point for soundings.—An initial point should be selected for soundings. The bridge should then be graduated on the side from which measurements of discharge are to be made, preferably the down-stream side, and the distances painted at regular intervals, the points generally used in making soundings and velocity observations being thereby indicated.

Cross-section.—The profile of the bed and banks to high water in the cross-section of the discharge measurements should be determined by using a level and a rod above water line and by soundings below.

Stage of no flow.—The stage of no flow should be determined, if possible, for use in constructing the rating curve. This will be the lowest point in the cross-section if the bed drops away rapidly below the station. In some places the low point in a bar below governs this stage. In others the ruling point is not evident.

Sketch of station.—A sketch should be made showing the structure from which gagings are to be made, the location of the gage and bench marks, the banks of the river for a considerable distance above and below the station, and any other features, such as tributaries, shoals, canals, dams, etc., that may affect the flow at the station.

CABLE STATIONS.

All the requirements for bridge stations apply also to cable stations, and in addition to these there are certain other requirements pertaining to the cable and its installation. The first step peculiar to the estab-

lishment of cable stations is the construction of the anchorages and supports, which should be installed before the cable is stretched.

If the anchorages are buried in the ground or hidden in any other way, auxiliary cables should be attached to them in order that the main cable and its connections may always be exposed for inspection.

The supports should be placed high, so that there will be no danger that floating ice or logs will strike the cable; also that measurements may be made at flood stages. The support on the bank of the stream where the car is kept may advantageously be about 5 feet lower than that on the other bank, so that the return to the initial point after the gaging is completed will be easier.

The measuring points should be determined by measurement from the initial point and marked by graduations painted on the cable.

The strength of the whole structure should be carefully considered, as upon it depend the lives of the engineers. The cost of the cable and its erection is so great that it is desirable that no part of its supports, anchorages, or equipment need be replaced for five years or more. It is therefore unwise to economize by using too little material in them. The cable and its appurtenances are described in detail on pages 30 to 33.

BOAT OR WADING STATIONS.

In the establishment of boat or wading stations the same general requirements apply as for a bridge station. In addition it is necessary to stretch one or more ropes or wires for holding the boat in position while making the measurements (Pl. VII, A). Boat stations can often be established to advantage at ferries where there are existing cables which may be used to keep the boat in position.

In making measurements at a wading station (Pl. V, A) it is necessary to stretch a tape across the stream to indicate the measuring points. Metallic tapes are best adapted to this use and are best held in position by iron rods with slits in the top.

SLOPE STATIONS.

If slope measurements of discharge are to be made, a location must be chosen having a straight channel from 200 to 1,000 feet in length, throughout which the cross-section and slope are reasonably uniform. The conditions of bed and banks should also preferably be permanent. The slope must be of sufficient magnitude to be measured without a large percentage of error.

As slope is the important factor in this method, the location of the gages by which it is to be measured should receive careful consideration. Theoretically the gages should be set in the current of the stream, which may, at high stages, be several tenths of a foot higher than water in the same cross-section near the banks. Such location of gages in the current is impossible unless bridges are available to which weight gages may be attached. In this case, the effect of bridge piers may be sufficient to vitiate the observations. Gages attached directly to bridge piers are not suitable for measurements of slope on account of the disturbance of the water around the pier. In locating gages on the banks the exposure should be the same for all gages to be utilized in conjunction for the determination of slope; otherwise the piling of water against one bank or its recession from another may indicate the slopes incorrectly.

At least two and preferably three gages should be constructed and placed in position at the ends (and in the center if three gages are used) of the course in which the slope is to be measured. The datum of each gage must then be accurately referenced by means of a permanent and easily accessible bench mark, and all gages connected by levels. Readings of the gages should be made to hundredths, and it will therefore be necessary to eliminate wave action by the use of some form of stilling box.

The accuracy of the estimates will depend largely on proper location of the gages, precision in the gage readings, and care in determining the levels connecting the gages.

The cross-section of the stream should be determined at each of the gages and careful notes made in regard to the character of the bed and banks of the stream. The determinations of these cross-sections may be made once for all at a low stage of the stream, if the bed and banks are permanent, or as often as may be necessary for good work if the conditions are changing.

THEORY AND PRACTICE OF MEASURING DISCHARGE.

The discharge at velocity-area stations is obtained by measurements of the area of the cross-section, which is determined by soundings, and of the velocity of the moving water, which is measured either directly by observation of the time of passage of a float over a measured course, or indirectly by noting the revolutions of the wheel of a current meter or by measurements of slope and the use of slope formulas. Discharge measurements are classed in accordance with these three methods of measuring velocity.

In making the measurement by means of a current meter or floats the area of the gaging section (fig. 8) that is perpendicular to the thread of the current of the stream is divided into partial areas for each of which the discharge is determined independently by multiplying its mean velocity by its area. The total discharge is the sum of the partial discharges. This computation of partial discharges eliminates the distribution of conditions existing in one part of the channel to parts in which they do not apply.

The procedure in case of slope measurements is somewhat different, as stated later.

AREA OF CROSS-SECTION.

The area of cross-section of a stream, the first factor in measuring discharge, depends on the contour of the bed, which is determined by soundings, and on the stage of the river, which is observed on the gage.

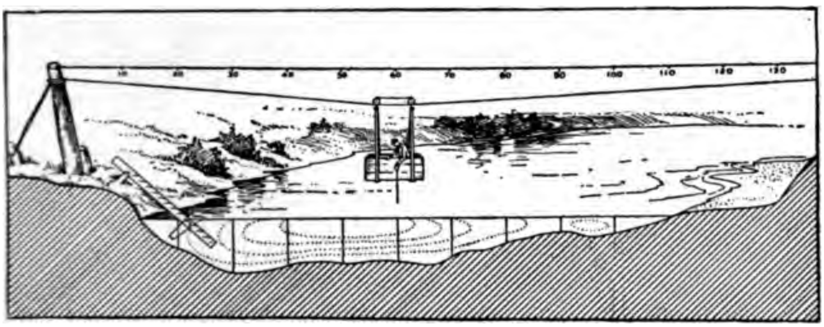


FIG. 8.—Cable Station and Section of River.

For current-meter stations the area of only the measuring section is required.

For float and slope stations the average area throughout the portion of the river used for the observations must be obtained. The areas at the ends of the section and at several intermediate points are measured and their average is assumed to be this mean area.

Soundings.—Soundings are made either by a graduated rod or by weight and line.

Sounding rods are limited in use to depths of less than 15 feet and are best adapted for use at wading and boat stations, where the depths and velocities are relatively small, but may occasionally be used at bridge stations where the bridge is not high above the water.

The weight and line are used in making soundings in water of greater depth than 15 feet, and from bridges or cables which are high above the water. Soundings from a bridge or cable with weight and line are most readily taken as follows: Lower the weight and line until the weight rests on the bed of the river directly underneath the measuring point. With the line taut, mark a point on it opposite a fixed point on the bridge or car; then raise the weight until it just touches the surface of the water and measure the length of the sounding line that passes the fixed point mentioned above. The depth is most readily measured by placing the end of a linen or metallic tape opposite the fixed starting point on the sounding line, grasping both the line and the tape in the hands, and drawing up the line and tape without permitting them to slip on each other until the weight rests on the surface of the water. The length of line thus drawn up, representing the depth of the water, can then be read directly from the tape. This measurement may usually be made by one person even when the depth is 10 to 12 feet. Where meter measurements are made from a bridge or cable, the meter cable, with meter and lead attached, is generally used for sounding if the depths and velocities are small, but care must be taken that the meter is not damaged.

The greatest and most common errors in measurements of discharge are caused by erroneous soundings. Errors in soundings by weight and line are due to the weight being carried downstream, so that it does not fall immediately below a point perpendicularly beneath the measuring point, or, sometimes, to the bowing of the line. Both these causes make the soundings too great. Errors in soundings with rods are due to the rod not being perpendicular, to the water rising on the rod, and to the rod sinking in the bed. In order to verify the accuracy of soundings made at medium or high stages they should be compared with those taken at low water.

Records of stage.—Records of stage may be made by means of either of the several kinds of gage already described, which should be read to tenths, half-tenths, or hundredths of a foot, the degree of accuracy of the reading depending on the size, stage, and slope of the stream.

If a recording gage is used, an attendant must visit it two or three times a week to insure a good and continuous record, and must at each visit or at least twice each week read a staff gage set to the same datum. The staff-gage reading with the date and time should be recorded on the graph, indicating by a cross its proper position on the curve. This precaution is essential in order that the datum of each record sheet shall be made to agree with that of the gage.

The datum of the non-recording gage must be frequently checked by comparison with a permanent bench mark. In case a record of stage is made from a non-recording gage an attendant or observer must read the gage daily or oftener if the stage is fluctuating rapidly or irregularly. Care and fidelity, with ability to read and write, are essential requirements for such observer.

Standard cross-section.—For gaging stations on streams whose beds are permanent or nearly so, a standard cross-section should be constructed from careful soundings. This cross-section should be referred to the zero of the gage, so that the depths for any stage can be found by adding the gage height to the depths below the zero of the gage. The soundings for the standard cross-section should be taken close enough together to develop all irregularities in the cross-section. Standard cross-sections have three uses: (1) They serve as checks on future soundings; (2) they indicate changes which may occur in the bed of the stream; and (3) they may be used in determining the area for measurements taken at times when it is impossible to make soundings on account of high water or other conditions.

VELOCITY.

Velocity of flowing water, indicated by V, is generally expressed in feet per second, and depends principally on (1) surface slope of the stream, (2) roughness of the bed, and (3) hydraulic radius.

The surface slope is the fall divided by the distance in which that fall takes place, and is represented by s. It depends on the slope of the bed, the channel conditions, and the stage. It is greater for a rising than for a falling stream.

The coefficient of roughness of the bed varies for different streams and stages of the water and is expressed by n.

The hydraulic radius or hydraulic mean depth is the area of the cross-section divided by the wetted perimeter. It is usually represented by R, and can be determined for all stages from a single complete measurement of permanent cross-section.

The mean velocity of a stream is the average rate of motion of all the filaments of water of the cross-section. It is not a directly measurable quantity, being usually found by dividing the total discharge by the area of the cross-section at a given stage. Its use is generally limited to purposes of comparison.

LAWS GOVERNING VELOCITY.

A systematic study of the flow of streams shows that mean velocity is in general a function of the stage and that the distribution of velocity through the cross-section follows well-defined laws which pertain to all streams flowing in open channels and which are in the main independent

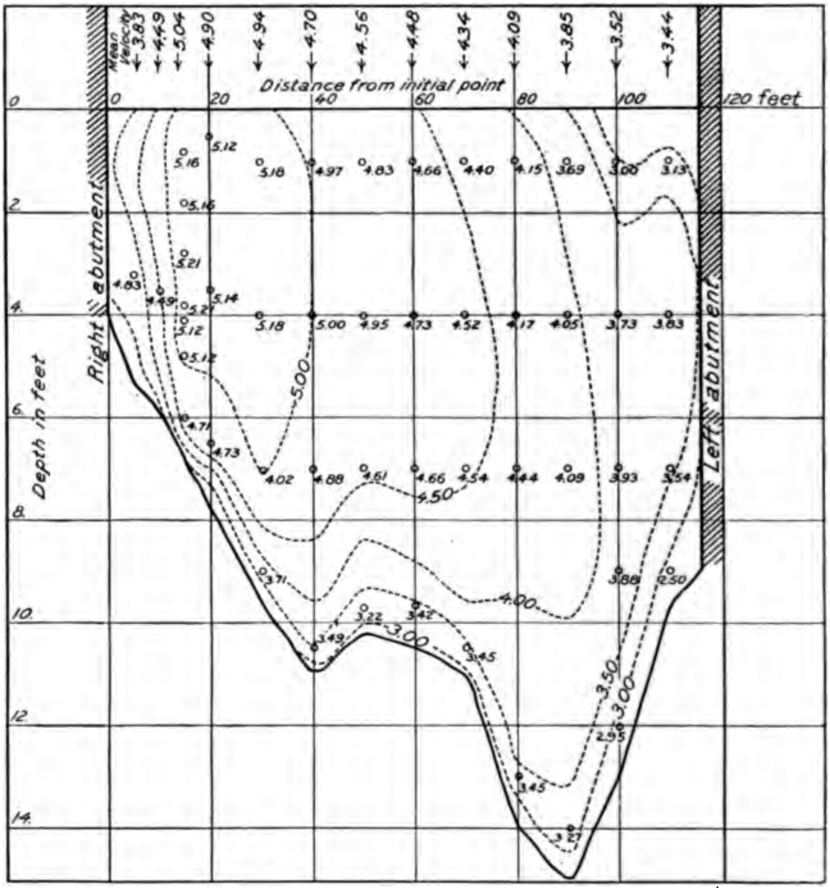


FIG. 9.—Cross-Section of Esopus Creek at Kingston, N. Y., showing Curves of Equal Velocity.

of the stage (figs. 9, 10, and 11). These laws make possible the determination of the velocity factor of the discharge measurement by comparatively few properly distributed observations of velocity. Upon them also depend the methods for determining the regimen of the stream.

These laws have been studied both mathematically and graphically

by means of vertical velocity-curves (figs. 10 and 11) which show graphically the distribution in a vertical line of the horizontal velocities of the filaments of water from the surface to the bottom of the stream.

Vertical velocity-curve.—A vertical velocity-curve is determined by a series of velocity observations taken at regular intervals in a vertical

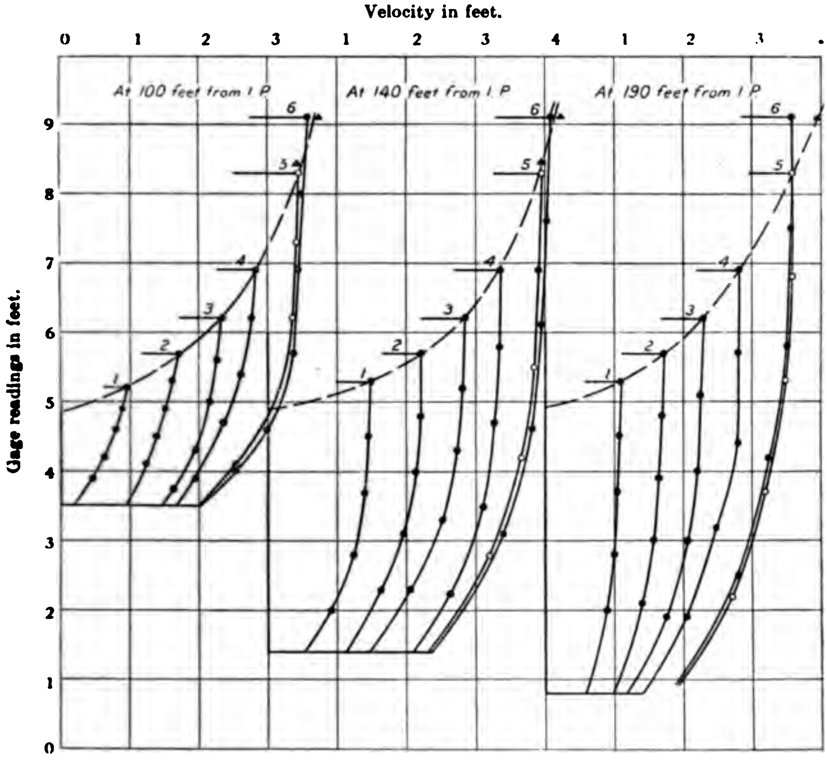


FIG. 10.—Groups of Vertical Velocity-Curves, Chenango River at Binghamton, N. Y.

from the surface to the bottom of the stream, usually from 0.5 to 1 foot apart. The results of these observations, when plotted with the velocities as abscissas and the depths as ordinates, define the curve.

Studies by Humphreys and Abbot[a] on the Mississippi, by General Ellis[b] on the Connecticut, and by the United States Geological Survey[c] on many streams under various conditions of depth, velocity, and

[a] Physics and Hydraulics of the Mississippi, 1851, p. 234.
[b] Report of the Chief of Engineers, U. S. Army, 1878, Part I, p. 259.
[c] Water-Supply and Irrigation Papers Nos. 95, 109, 187 and others.

roughness of bed, show that these vertical velocity-curves have approximately the form of the parabola whose axis, coinciding with the filament of maximum velocity, is parallel with the surface and is in general situated between the surface and one-third of the depth of the water. From the maximum the velocity decreases gradually upward to the surface and downward nearly to the bottom, where it changes more rapidly on account of the friction on the bed. As the depth and velocity increase, the curve approaches a vertical line as its limiting position.

Distribution of velocity in the vertical.—If, as stated above, the velocities in a vertical line vary as the ordinates of a parabola, it may be shown mathematically that (1) a filament of water which has the same velocity as the mean of the velocities in that vertical occurs at a point between .5 and .7 of the depth measured from the surface of the stream, and (2) that the mean velocity equals the mean of the velocities occurring at .2114 and .7886 of the depth.

The demonstration[a] of the location of the filament of mean velocity is based on the theory of mean values, using the fundamental equation,

$$y = ad + d\sqrt{\tfrac{1}{3}(a^3 + b^3)},$$

in which d equals the total depth of water; a, the depth of maximum velocity below the surface; b, the unitary complement of a; y, the depth of the thread of mean velocity.

By assigning values to a between 0 and $\frac{1}{3}d$ and substituting them with simultaneous values of b in the above equation, there results the following table, showing the depth of the mean ordinate for parabolic curves with various positions of depth to the maximum ordinates.

Depth of maximum ordinate.	*Depth of mean ordinate.*
When $a = 0$	$y = 0.58d$
$a = 0.10d$	$y = 0.59d$
$a = 0.15d$	$y = 0.60d$
$a = 0.20d$	$y = 0.62d$
$a = 0.25d$	$y = 0.63d$
$a = 0.30d$	$y = 0.65d$
$a = 0.33d$	$y = 0.67d$

The maximum ordinate in streams that are neither very shallow nor very deep usually lies at or above one-third depth (see table, pp. 50-51).

[a]Engineering News, Vol. LV, p. 47.

If it lies above one-fourth depth, the ordinate at 0.6 depth is very closely the mean ordinate. If the stream is very deep the maximum thread lies generally at a greater proportional depth, and the thread of mean velocity therefore lies at a greater depth. If a stream is shallow and has in addition a rough bed, the frictional effect on the flow is so large that the vertical velocity-curve is no longer parabolic near the bottom and the thread of mean velocity may be near mid-depth.

A study of vertical velocity-curves shows that the mean velocity in the vertical equals from 85 to 95 per cent of the surface velocity, and it also equals one-fourth the sum of the velocity near the surface plus twice the velocity at mid-depth plus the velocity near the bottom.

That these properties generally hold in nature has been proved by hundreds of vertical velocity-curves made on a large number of streams having a wide range in conditions of depth, character of beds, and magnitude of velocity. Fig. 10 shows the form of a number of typical vertical velocity-curves and the table on pages 50–51 gives a summary of results of a large number of vertical velocity-curves.

A study of these measurements, together with many others which are not available for publication, shows the general applicability in nature of the foregoing laws upon which depend the common methods of measuring discharge by floats and current meters.

METHODS OF DETERMINING MEAN VELOCITY IN A VERTICAL.

In the application of the laws of the distribution of velocity there have been developed the following methods of determining mean velocity in the vertical: (1) Vertical velocity-curves; (2) .6 depth; (3) surface; (4) .2–.8 depth; (5) three-point; (6) integration.

Measurements of velocity are therefore generally made by one of the above-mentioned methods, each of which has its special advantages and limitations. Their essential use is to determine the mean horizontal velocity in a vertical line and not features of measurement or computation that involve other factors.

As a discharge measurement contains a large number of velocity determinations the error introduced in the result by an individual erroneous measurement of velocity is generally inappreciable.

A description of the methods and a statement of the relative accuracy obtained by each, as determined by a comparative study of all available vertical velocity-curves, will now be given.

Vertical velocity-curve method.—By the vertical velocity-curve method **measurements of horizontal velocity** are usually made just under the

Summary of vertical velocity-curves in open channels.

Stream and locality.		Number of curves.	Range of depth—feet.	Range of velocities—feet per second.	Depth thread mean velocity in per cent of total depth.	Depth thread maximum velocity in per cent of total depth.	Coefficients for reducing to mean velocity. Six-tenths depth.	Mid-depth.	Top.	Mean of top and bottom.	$\frac{.2+.8}{2}$	$\frac{T+2M+B}{4}$
Measurements since 1905.												
Ohio	Wheeling, W. Va	14	5.3–20.8	.92–4.43	66	13	.98		.93		1.005	
Monongahela	Lock No. 4, Pa	7	10.9–17.5	.74–1.13	68	0	.96		.85		.990	
St. Mary	Cardston, Alberta	8	2.7– 3.9	3.78–6.32	58		1.01		.88		1.001	
Jefferson	Sappington, Mont	7	2.7– 5.7	1.78–3.45	63		.97		.84		.997	
Gallatin	Logan, Mont	7	3.7– 5.5	.88–3.85	62		.98		.85		.991	
Missouri	Cascade, Mont	22	2.1–12.0	.79–3.04	62		.99		.88		1.002	
Marias	Shelby, Mont	20	1.9– 4.2	.57–4.52	60		1.01		.86		1.004	
Yellowstone	Glendive, Mont	11	4.3– 8.5	2.42–3.79	60		1.00		.89		1.003	
Bighorn	Fort Custer, Mont	8	3.9– 7.4	1.22–2.60	59		1.00		.86		.998	
Shoshone	Cody, Wyo	16	4.3– 7.3	4.93–8.45	58	13	1.02	.96	.87		1.00	
North Platte	Saratoga, Wyo	8	3.9– 5.1	1.82–3.61	63	26	.98		.94	1.31	1.026	1.09
Pecos	Dayton, N. Mex	10	4.1– 8.2	1.68–2.81	58		1.03		.82		1.003	
Pecos	Pecos, Tex	6	2.2– 6.0	1.52–3.06	63		.97		.98		1.004	
Green	Greenriver, Wyo	36	2.2– 6.4	1.05–4.12	58	14	1.01		.90	1.20	1.026	1.06
Yampa	Steamboat Springs, Colo	8	4.2– 5.2	7.13–8.42	59	7	1.01		.83	1.10	1.003	1.02
Grand	Hot Sulphur Springs, Colo	6	2.8– 8.2	.89–2.87	63	23	.98		.85	1.30	1.01	1.03
Grand	Kremmling, Colo	15	9.2–27.5	1.11–4.02	65	24	.97		.93	1.16	1.001	1.03
Gunnison	Cimarron, Colo	9	4.3–15.5	3.83–8.70	64	19	.99		.92	1.06	1.003	1.02
Gunnison	Cory, Colo	5	4.9–10.4	1.63–9.59	63	17	.99		.92	1.09	1.007	1.01
Big Blackfoot	Bonner, Mont	7	4.2– 6.8	1.41–2.49	62		.98		.85		1.016	
Clark Fork	Missoula, Mont	5	5.4– 7.6	1.72–2.41	60	10	1.00		.79		.997	
Yakima	North Yakima, Wash	7	4.9– 7.6	1.58–2.56	59		1.01		.83	1.06	.999	1.00
Yakima	Yakima, Wash	50	3.2–12.6	.65–8.56	59		1.01		.85	1.04	1.003	.99
Yakima	Kiona, Wash	20	2.0– 6.1	.25–3.24	61		.98		.84	1.05	1.00	.98
Kachess	Easton, Wash	5	3.4– 4.3	1.44–4.78	61		.99		.85	1.04	1.009	.99
Clealum	Roslyn, Wash	20	4.7–11.7	.37–3.89	58		1.01		.84	1.09	.995	1.02
Naches	North Yakima, Wash	25	2.4– 6.0	.65–7.38	58		1.01		.85	1.04	.998	.99
Snake	Minidoka, Idaho	14	4.0–10.7	1.13–3.05	62	20	.99		.89		.994	
Big Lost	Mackay, Idaho	7	2.4– 3.7	1.23–3.85	71	42	.95		.95		.993	
Palouse	Hooper, Wash	28	1.6– 8.0	.97–6.49	60		1.00		.84	1.06	1.004	1.00
John Day	McDonald, Oreg	15	3.6– 5.9	1.87–4.44	65		.98		.89			
Umatilla	Pendleton, Oreg	13	2.2– 7.5	.47–2.73	64		.97	.90	.87		.970	
Deschutes	Lava, Oreg	10	3.0– 4.1	.88–1.98	66		.99	.94	.92		.994	
West Fork Deschutes	Lava, Oreg	27	1.7– 7.1	.58–2.65	64		.98	.96	.89		.992	
Mean of 476 curves					62		.99		.87		1.001	
Highest			27.5	9.59	71		1.03		.98		1.026	
Lowest			1.6	.25	58		.95		.79		.970	

Measurements prior to 1905.												
Susquehanna	McCall Ferry, Pa	73	3.3–30.0	1.21–5.80	68		.94		.92	1.07		
Susquehanna	McCall Ferry, Pa	68	5.0–36.0	1.40–9.70	73		.97		.90			
Susquehanna	Harrisburg, Pa	20	3.2–8.0	1.52–2.71	61		1.01		.85	1.08		
Susquehanna	Binghamton, N. Y	36	2.5–8.1	.80–4.86	61		.99			96		
Chenango	Binghamton, N. Y	34	1.7–7.4	.46–3.38	62		.98			1.04		
Catskill Creek	South Cairo, N. Y	47	2.1–7.0	.27–3.91	58		1.02		.83			
Delaware (E. Br)	Hancock, N. Y	6	2.1–4.8	.75–4.12	59		1.02		.82			
Delaware (W. Br.)	Hancock, N. Y	12	1.7–4.2	1.67–4.45	58		1.02		.83			
Esopus Creek	Kingston, N. Y	53	1.9–14.0	.65–5.04	62		1.00		.88			
Esopus Creek	Olivebridge, N. Y	68	0.9–5.2	.42–7.62	58		1.01		.69			
Fishkill Creek	Glenham, N. Y	22	2.1–5.0	.26–1.75	59		1.04		.79			
Housatonic	Gaylordsville, Conn	36	2.1–6.2	.67–3.66	58		1.01		.83			
Rondout Creek	Rosendale, N. Y	21	4.4–8.2	.35–2.55	61		.99		.84			
Schoharie Creek	Prattsville, N. Y	11	1.1–6.6	.52–3.20	58		1.01		.80			
Tenmile	Dover Plains, N. Y	33	1.8–7.3	.24–2.15	61		.99		.88			
Wallkill	Newpaltz, N. Y	52	3.3–17.3	.18–4.72	64		.98		.85			
Wappinger Creek	Wappinger Falls, N. Y	17	2.5–4.7	.58–2.11	58		1.03		.78			
Appomattox	Mattoax, Va	8	2.5–4.9	.92–2.30	62		1.00		.81	1.11		
James	Cartersville, Va	6	3.0–4.4	1.88–2.37	60		1.01		.88	1.08		
Roanoke	Roanoke, Va	8	1.8–3.3	.87–1.96	61		.99		.81	1.01		
Staunton	Randolph, Va	20	2.0–6.0	1.35–3.11	66		.96		.95	1.12		
Dan	Madison, N. C	42	1.3–3.8	.73–3.00	62		.99		.86	1.01		
Dan	South Boston, Va	13	3.5–9.0	.90–2.41	64		.98		.90	1.09		
Reddie	North Wilkesboro, N. C	3	1.7–2.5	1.62–2.01	58		1.02		.83	.97		
Yadkin	North Wilkesboro, N. C	33	2.7–6.4	1.20–3.16	66		.96		.91	1.12		
Yadkin	Salisbury, N. C	10	3.1–11.3	.96–3.23	59		1.02		.81	1.06		
Catawba	Morgantown, N. C	39	1.9–5.0	.74–2.84	61		1.00		.82	1.04		
Catawba	Mount Holly, N. C	2	6.3–7.0	.49–.74	58		1.00		.88	1.08		
Catawba	Belmont, N. C	1	3.5	1.54	59		1.01		.80	1.05		
Wateree	Camden, S. C	4	12.3–17.7	1.33–1.83	58		1.01		.90	1.13		
Broad	Alston, S. C	4	5.0–8.9	2.79–4.00	65		.96		.89	1.22		
Saluda	Waterloo, S. C	3	3.0–8.5	1.76–2.01	62		.97		.82	1.03		
Watauga	Elizabethton, Tenn	10	2.6–5.3	.34–1.01	61		.98		.82	1.07		
South Fork Holston	Bluff City, Tenn	13	1.9–4.1	1.08–6.77	62		1.02		.80	1.05		
Nolichucky	Greeneville, Tenn	22	1.6–5.1	.49–2.69	59		1.02		.80	1.02		
French Broad	Oldtown, Tenn	9	4.2–8.0	.53–2.18	62		.99		.86	1.07		
Pigeon	Newport, Tenn	4	4.4–11.6	.82–1.14	61		.99		.91	1.15		
Tuckaseegee	Bryson City, N. C	36	3.5–8.5	.50–3.39	64		.97		.93	1.09		
Little Tennessee	Judson, N. C	11	2.7–6.0	3.84–7.45	59		1.03		.83	1.06		
Mean of 910 curves					61		1.00		.85	1.07		
Highest			36.0	9.70	73		1.04		.95	1.22		
Lowest			1.1	0.18	58		.94		.78	.96		
Shallow streams with rough beds.												
St. Mary	Dam Site, Mont	24	1.8–4.2	2.60–6.36	60	9	1.00	.93	.86		1.01	
Swiftcurrent Creek	Babb, Mont	16	1.6–3.2	3.28–6.81	57	0	1.03	.96	.87		1.004	
South Fork Shoshone	Marquette, Wyo	20	1.8–3.6	3.61–6.79	58	0	1.02	.93	.82		1.016	
Ana	Summer Lake, Oreg	6	1.8–2.4	1.83–3.37	63	10	.97	.92	.89		1.00	
Tieton	North Yakima, Wash	14	1.8–4.5	4.42–8.04	56	0	1.04	.96	.80		1.00	.98
Wallowa	Wallowa, Oreg	4	2.2–2.9	4.16–4.93	58	0	1.04	.94	.78		1.004	
Kruzgamepa	Salmon Lake, Alaska	108	1.6–3.6	1.43–4.70	64	23	.97	.91	.86		1.004	.99
Small streams	Seward Peninsula, Alaska	27	1.7–2.5	2.15–3.47	61	14	1.00	.93	.84		1.004	1.00
Mean of 219 curves					60	7	1.01	.92	.84		1.005	.99
Highest			4.5	8.04	64	23	1.04	.96	.89		1.016	
Lowest			1.6	1.43	.56	0	.97	.91	.78		1.00	

surface, at .5 foot below the surface, and at each fifth to each tenth of the depth, from the surface to the bed of the stream. These measured velocities, when plotted, define for each such observation point the vertical velocity-curve from which the mean velocity in that vertical is determined.

In computing the mean velocity, the velocity observations are plotted with depths as ordinates and velocities as abscissas, and a mean curve is drawn through these points. The quotient obtained by dividing the area bounded by this curve and its axis by the depth is the mean velocity. In the absence of a planimeter for measuring the area, the depth should be divided into ten or more equal parts. The mean of the center ordinates of the parts equals nearly the mean velocity.

For purposes of study vertical velocity-curves are sometimes plotted (fig. 11) with per cent of mean velocity as abscissas and corresponding depths expressed as percentages of the total depth as ordinates. The resulting curves are of course distorted, but are useful in determining mean curves.

The vertical velocity-curve method is valuable as a basis for comparison of all other methods, for determining coefficients to be used in reducing values obtained by other methods to the true value, for use under new and unusual conditions of flow, and for measurements under ice. The method is not, however, in general use for making observations of velocity for routine discharge measurements, because the increased accuracy thereby obtainable is frequently overbalanced by errors arising from changes in stage of the stream during the longer time required for the measurement.

In making observations of velocity for the construction of vertical velocity-curves, velocities should also be measured at .2, .6, and .8 depth, in order that the mean velocity determined by methods in which these depths are used can be directly compared with that determined by the vertical velocity-curve method.

Vertical velocity-curves should be constructed for all stations at different stages in order to determine whether coefficients should be applied to the results obtained by the other point methods. Such application of coefficients should be made, however, only on unquestionable evidence furnished by a large number of vertical velocity-curves. The coefficient deduced from a single curve is rarely applicable to the entire cross-section of the stream.

The coefficients as determined by vertical velocity-curves for reducing the velocity by either of the other point methods to mean velocity

may be plotted with stage as the other ordinate and thus determine a curve which may define the coefficient to be used at any stage.

The .6 depth method.—In practical measurement of stream discharge it is necessary to determine the horizontal velocity in a large number of

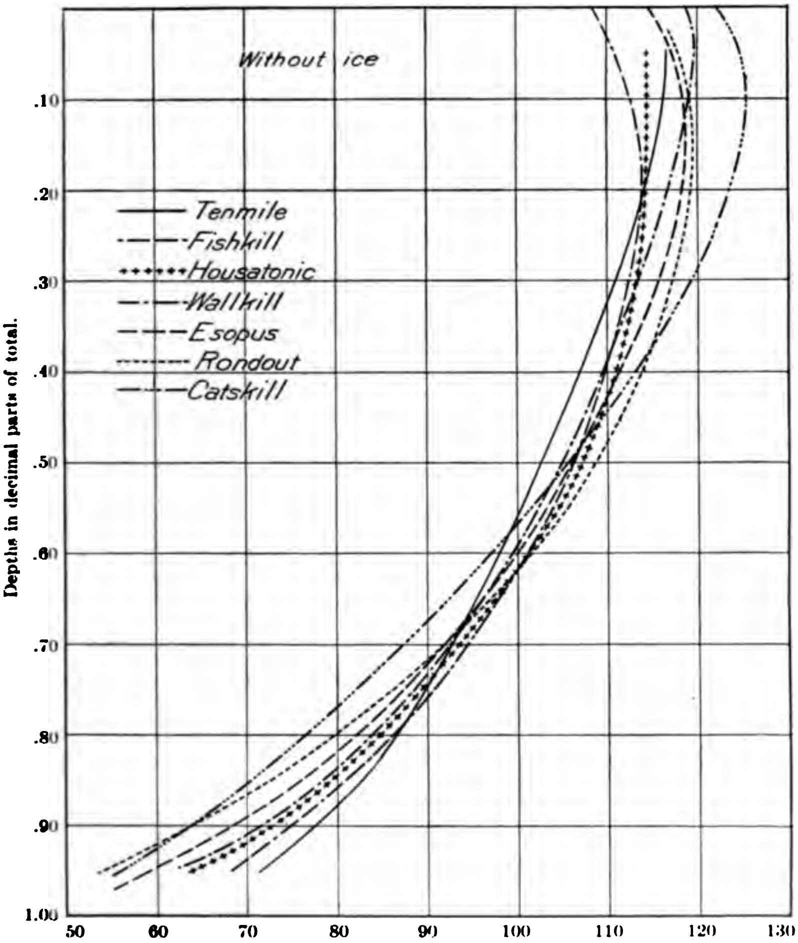


FIG. 11.—Mean Vertical Velocity-Curves for New York and Connecticut Rivers.

verticals. Therefore, a method must be used which requires not more than three velocity observations in each vertical. If one point is used it is desirable that it be in such position that the use of a coefficient is not necessary to determine the mean velocity. The foregoing theory

shows that the .6 point lies approximately at .6 depth of the stream. The preceding table shows that the thread of mean velocity lies between 54 per cent and 7[illegible] per cent of the depth, with an average of 61 per cent. The error resulting from the use of .6 depth is very small, ranging from [illegible] per cent to [illegible] per cent, with a mean of [illegible] per cent. Therefore, in the .6 depth method it is assumed that the velocity at .6 depth is the mean velocity in the vertical and the meter is held at that point in this method.

Although this method is intended to be used without coefficients it may be found by vertical velocity-curve measurements that a coefficient is necessary in some instances to reduce the observed velocities to the mean. The method is applicable over a wide range of conditions, is easy of execution and is reasonably accurate for normal flow in the straight reaches of all streams except very deep and very shallow ones.

The surface method.—The surface method is used in the measurement of velocities of swift streams, especially at times of freshet, when it is impracticable to sink the meter much below the surface. Therefore the observation of velocity is made at a point near the surface, but far enough below to eliminate any disturbance from wind or waves. The point of observation in this method should be from .5 foot to 1 foot below the surface, its location depending on the depth of the stream. The measured velocity must, however, be multiplied by a coefficient to reduce it to the mean. This coefficient, as shown in the preceding table, varies between 78 and 98 per cent, depending upon the depth of the stream and the magnitude of the velocity. The deeper the stream and the greater the velocity the larger the coefficient. For average streams in moderate freshet a coefficient of about 90 per cent will generally give fairly accurate results. In flood work coefficients of 90 to 95 per cent should be used.

The two-point method.—The two-point method is used on streams in which the location of the point of mean velocity is uncertain, or when greater accuracy is desired than can be obtained by the .6 depth method. As noted in the foregoing theory, the mean of the velocities at .2 and .8 depth gives nearly the mean velocity in the vertical. The preceding table shows that this theory holds very closely in nature. Therefore in this method the meter is held at .2 and .8 depth of each vertical.

Observations of velocity near the surface and near the bottom of the stream have in the past been used in the two-point method. Both the theory and the tables show that .2 and .8 depth should be used,

The three-point method.—The three-point method approaches more nearly the vertical velocity-curve and is used for obtaining greater accuracy than is possible by the one- and two-point methods. In this method the meter should be held at .2, .6, and .8 depth. The mean velocity is then obtained by dividing by 4 the sum of the velocities measured at .2 and .8 depth plus 2 times that at .6 depth.

In this method the observations have in the past been frequently taken at top, bottom, and mid-depth, but both theory and experience show that .2, .6, and .8 depth are the proper points for such observations.

The integration method.—The integration method is used both for obtaining the mean velocity in the vertical and also the mean velocity in the entire cross-section of the stream.

In determining the mean velocity in the vertical the meter is moved at a uniform speed from the surface of the water to the bed of the stream and return and the revolutions and time are observed. The meter thus passes successively through all velocities in that vertical and the resulting observations determine the mean in that vertical. The method is valuable for checking other methods, but generally requires the service of at least one more man to observe time, as the engineer must be occupied with the movements of the meter. It is consequently not so commonly used as the point methods. The Price meter is not suited to observations by this method, as the vertical motion of the meter causes the wheel to revolve. The Haskell meter, on the other hand, may be moved vertically, swiftly or slowly, with no effect on the wheel.

In determining the mean for the entire section the meter is moved with uniform speed throughout the section, usually in a zigzag path extending from surface to bottom and from side to side of the section.

DISCHARGE MEASUREMENTS.

As previously stated, the discharge measurement of a stream consists in determining the area of its cross-section and the velocity of its moving water. A general discussion of the laws governing and methods of determining these two factors has already been given. The application of these methods of determining area and velocity in the various kinds of discharge measurements needs further explanation, and the methods used will depend primarily on whether the velocity is determined by current meter, float, or slope.

CURRENT-METER MEASUREMENTS.

Procedure.—In making a current-meter measurement the cross-section (figs. 8 and 12) is divided into partial areas, varying in width from 2 to 20 feet, depending on the size of the stream. These partial areas are bounded by perpendiculars terminating at points in the surface known as measuring points, because they indicate where the observations of depth and velocity are taken. They should be so spaced as to show any irregularities either in the cross-section or the velocity. When measurements are made at bridge or cable stations, the measuring points should be permanently marked on the bridge rail or floor, or on the cable, and used for successive measurements of discharge. When measurements are made at boat and wading stations the points will be indicated by the graduations on a tape or tagged line, which is generally stretched at the time of each measurement.

The procedure in the measurement will vary somewhat, depending on the sounding appliance. If the meter and cord are used for sounding, observations of depth and velocity will be made at each measuring point successively across the stream. If other sounding apparatus is used, soundings will be made at all measuring points prior to taking the velocities.

In making velocity observations, one of the methods described on pages 49–55 should be used, the method chosen depending upon the conditions at the station. Care must be taken to place the center of the meter wheel at the points called for by the method. This is best accomplished by measuring the required depth on the meter line with the wheel in the surface of the water, and then lowering the meter into position. Special attention is called to the requirement, both in sounding and in placing the meter in position for observing velocity, that a tagged line should not be used for measuring depth. Such distances should be determined by means of a tape line, as indicated on page 44. In making the observations a stop-watch is desirable but not indispensable. In general, time should be noted at the click of the receiver, or at the start or finish of the buzz. The number of revolutions indicated by the receiver or buzzer, not including the buzz at the start, should then be counted for a period of 100 seconds, the count also at the 50-second interval being noted as a check on the total count. These particular time intervals are used instead of one and two minutes on account of the ease in converting observed revolutions into revolutions per second by the movement of the decimal point.

In case the velocity of the current makes other than a right angle with the measuring section the deviation from the right angle must be observed and a coefficient applied to reduce the velocity to the normal. This coefficient can usually be applied to the final completed discharge. In case, however, the angle varies throughout the cross-section, it is necessary to apply appropriate coefficients to the various observed velocities. The angle can readily be determined by holding the meter just below the surface of the water and placing the notebook perpendicular to the cross-section of the stream and drawing a line parallel to the meter. This line should be divided into ten arbitrary divisions and projected upon a line normal to the gaging section. The length of this projection will be the coefficient to be used.

If the current-meter measurement of discharge is made at a regular gaging station established for obtaining a record of discharge, a certain routine should be followed, consisting of the steps indicated below in consecutive order.

1. Measure and record the length of the chain, if a chain gage is used.
2. Correct chain length if necessary and record changes.
3. Read the gage.
4. Set up meter; test it to determine if it is working properly; see that buzzer or receiver is in order.
5. Make the observations necessary for the measurement of the discharge by one of the methods described on preceding page.
6. Read the gage.
7. Check over the notes to make certain that all records have been made.
8. Wash thoroughly the cell of the buzzer, if used, and clean and put up the meter.
9. Be careful to note all "remarks" relative to changes in stage, backwater, wind, etc., that may be of future value.

Each of the above observations is essential to the reliability of the record at a station. On each visit of the engineer the stage of the river should also be determined by observing the distance to the surface of the water from a reference point as a check on the gage record. Both the gage reader and his record should be seen if possible, and his attention called to any lack of interest or apparent discrepancies in his work.

Computations.—Four methods of computing discharge measurements from observed velocities and depths are in general use. These are shown by the following eight formulas, which refer to fig. 12.

Formulas for computing mean depth, mean velocity, area, and discharge

$$Q = a \cdot V_a L \tag{1}$$

$$Q = \frac{(a + 6b + c)}{8} L V_b \tag{2}$$

$$V = \frac{(V_a + V_b)}{2} \tag{3}$$

$$d = \frac{a + b}{2} \tag{4}$$

$$Q = d V L = \left(\frac{a + b}{2}\right) L \left(\frac{V_a + V_b}{2}\right) \tag{5}$$

$$d'_m = \frac{a + 4b + c}{6} \tag{6}$$

$$V'_m = \frac{V_a + 4V_b + V_c}{6} \tag{7}$$

$$Q' = d'_m V'_m 2L = \left(\frac{a + 4b + c}{6}\right) 2L \left(\frac{V_a + 4V_b + V_c}{6}\right) \tag{8}$$

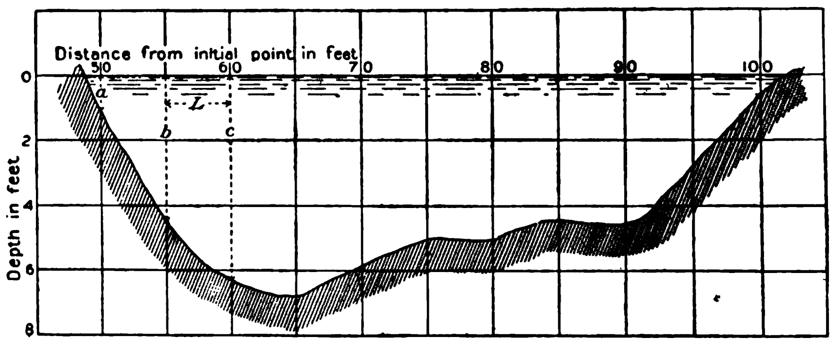


FIG. 12.—Typical Cross-Section to Illustrate Computation of Discharge Measurements.

d = mean depth for a single strip.
V = mean velocity for a single strip.
d'_m = mean depth for a double strip.
V'_m = mean velocity for double strip.
a, b, c, are three consecutive depths, L feet apart.
V_a, V_b, V_c, are observed velocities in the verticals a, b, c.
L = the width of a single strip.
Q' = the discharge through double strip.
Q = discharge through single strip.

These methods are similar in the following particulars: The measuring section is divided into elementary strips; the mean velocity, area, and discharge are determined separately for either a single or double strip; the total discharge and area are found by summing those for the various strips; and the mean velocity is found by dividing the total discharge by the total area.

Equation (1) outlines the easiest and in some respects the best method. In it the velocities and depths are taken at the middle points of the strips. If the strips have equal widths the partial areas and discharges are not generally computed as such, but, instead, the sum of the products of depths and velocities is multiplied by the width of the stream, which product determines the total discharge. This method, while open theoretically to minor criticism, is never far wrong in its results. It was used by General Ellis in his work on Connecticut River. If employed, however, it is desirable that the total area of cross-section of water shall be determined for use in studying sources of error in the resulting discharge and in discordant measurements.

Equation (2) represents a modification of the above method, in which each observed velocity is applied to an area extending on either side halfway to the next observing point. The expression $\frac{(a + 6b + c)}{8}$ for the mean depth of that area is based on the assumption that the bed of the stream is composed of straight lines joining the points of sounding. This method is theoretically somewhat more accurate than the former but is more difficult of application.

Values of mean depth may be determined mentally with considerable rapidity as follows: Add algebraically, to the center depth, one-eighth of the algebraic sum of the differences between the center depth and the two adjacent depths.

Example: $a = 4.6$	$4.6 - 5.8 = -1.2$
$b = 5.8$	$8.4 - 5.8 = +2.6$
$c = 8.4$	$-1.2 + 2.6 = +1.4$
	$+1.4 \div 8 = +0.175$

$$d_m = 5.8 + (+.175) = 5.975$$

Proof of method:

$$\frac{a + 6b + c}{8} = \frac{(a - b) + 8b + (c - b)}{8} = b + \frac{(a - b) + (c - b)}{8}.$$

The method of computation outlined in equations (3), (4), and (5) differs from that contained in equation (2) in that both adjacent velocities and depth are averaged. The labor of computation is thereby somewhat increased, although not materially, while the results are probably no more nearly correct, and are possibly open to some question on account of such application of averaged velocities to a considerable area. This method is also used in computing odd-length strips occurring at piers or at the banks. These equations are based on the assumption that the velocities and depths vary as the ordinates to straight lines.

The method outlined in formulas (6), (7), and (8) is more laborious in its application than are the other methods indicated. By it the discharge through two consecutive strips is determined. It is based on the theory that the bed of the stream is approximately parabolic in form, and that the horizontal curve of velocities is parabolic.

On page 61 is a sample of notes and computations for a current-meter measurement.

FLOAT MEASUREMENTS.

In measuring velocity by a float the time required for its passage over a given distance is observed. The quotient obtained by dividing this distance by the time is the velocity of that portion of the stream traversed by the float.

The first step in making a float measurement is to select and measure the "run" over which the floats are to pass. The ends of the "run" should then be definitely marked on one or both banks of the stream, either by tagged lines or by range poles or signals, which are used to determine also the positions of the floats in the channel. The type of float used will depend upon local conditions of channel and current. Tube floats are generally limited in use to artificial channels. Subsurface floats are used only in deep streams, and their practicability is doubtful on account of the uncertainty of the position of the float. Surface floats are adapted for general use under all conditions. Velocities observed by them must, however, be reduced by a coefficient. The magnitude of this coefficient varies between .85 and .95, depending upon the stage and character of the stream, as explained on page 54.

In a discharge measurement by this method a number of velocity determinations should be made at varying distances from the shore. From them the mean velocity of the whole section can be determined by plotting the mean position of each float, as indicated by its average distance from the banks, as an ordinate, and the corresponding time

Gaging made June 8, 1906, *by Follansbee and Padgett, on Roanoke River at Randolph, Va. Gage height in ft.: beginning* 4.20, *end* 4.20, *mean* 4.20. *Meter No.* 130. *Total area,* 727 *sq. ft. Mean velocity* 1.93 *ft. per sec. Discharge* 1401 *sec.-ft.*

Distance from initial point.	Depth.	Observations.				Velocity computations.				Computations of area of section.			Discharge of section.	Remarks.
		Depth of observations.	Time in seconds.	Revolutions.		Total number revolutions.	Revolutions per second.	Velocity per second.	Mean velocity per second.	Width.	Mean depth.	Area.		
178	0													Water's edge, right bank.
180	1.4	.8	50	27	27	54	.54	1.22	.61	2	.70	1.4	0.9	
185	2.2	1.3		27	28	55	.55	1.24	1.31	10	2.33	23.3	30.5	
190	3.8	2.3		37	37	74	.74	1.67						
200	3.9	2.3		44	45	89	.89	2.01	1.95	20	3.88	77.6	151.3	Sine of angle of current with bridge = .95.
210	3.9	2.3		45	44	89	.89	2.01						
220	3.7	2.2		50	51	101	1.01	2.28	2.26	20	3.63	72.6	164.1	
230	3.1	1.9		53	54	107	1.07	2.41						Note: Computed by formulas 5 and 8, page 58.
240	2.5	1.5		60	61	121	1.21	2.73	2.60	20	2.63	52.6	136.8	
250	2.7	1.6		51	51	102	1.02	2.30						
260	2.5	1.5		54	54	108	1.08	2.44	2.47	20	2.60	52.0	128.4	
270	2.9	1.7		61	61	122	1.22	2.75						
280	3.0	1.8		60	60	120	1.20	2.71	2.71	10	2.85	28.5	77.2	
285	2.2	1.3		59	60	119	1.19	2.69	2.60	10	2.52	25.2	65.5	
290	3.3	2.0		46	47	93	.93	2.10						
295	0							1.00	1.55	5	1.65	8.2	12.7	Face pier. Velocity estimated.
Pier.														
315	3.6	2.2		26	26	52	.52	1.17						Face pier.
320	5.0	3.0		48	48	96	.96	2.16	1.66	5	4.30	21.5	35.7	
330	3.0	1.8		54	54	108	1.08	2.44	2.40	20	3.47	69.4	166.6	
340	3.8	2.3		54	55	109	1.09	2.46						
350	2.8	1.7		47	47	94	.94	2.12	2.24	20	3.28	65.6	146.9	
360	4.7	2.8		55	55	110	1.10	2.48						
370	3.0	1.8		8	8	16	.16	.40	1.08	20	3.45	69.0	74.5	
380	4.0	2.4		53	53	106	1.06	2.39						
390	2.8	1.7		44	44	88	.88	1.99	2.08	20	3.23	64.6	134.4	
400	4.2	2.5		47	46	93	.93	2.10						
410	2.9	1.7		42	41	83	.83	1.87	1.88	20	3.22	64.4	121.1	
420	3.5	2.1		37	37	74	.74	1.67						
425	2.6	1.6		23	23	46	.46	1.03	1.03	10	2.65	26.5	27.3	
430	2.0	1.2		8	8	16	.16	.40						
435	0							0	.20	5	1.00	5.0	1.0	Water's edge, left bank.
Total												727.4	1474.9 × .95 = 1401.	

of the run as an abscissa. A curve through the points so located shows the mean time of run at any point across the stream. Velocities for each partial area of cross-section are scaled from this curve, reduced to feet per second, and multiplied each by its area to determine partial discharge. The sum of the partial discharges is the total discharge. The distances should be expressed in feet and the time in seconds. A stop-watch is necessary for the satisfactory determination of the time required by the floats to pass over the course.

The area used in float measurements is the effective area of the section of the river over which the runs are made and is determined by averaging the areas of cross-section of the stream measured at the ends and at intermediate points.

SLOPE MEASUREMENTS.

The mean velocity of a stream has been expressed in the Chezy formula as $V = c\sqrt{Rs}$, in which c is the coefficient combining the total effects of roughness of the bed and all other conditions which may affect the velocity, except the slope and hydraulic radius. This formula has long served as a nucleus about which slope data have been collected, and has been used as a basis for work by Kutter,[a] who developed the following expression for the value of the coefficient c in terms of s, R, and n.

$$c = \frac{41.6 + \frac{.00281}{s} + \frac{1.811}{n}}{1 + \left\{41.6 + \frac{.00281}{s}\right\}\frac{n}{\sqrt{R}}}$$

This, when introduced into the original formula, gives

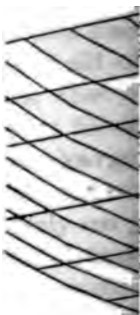
$$V = \left\{\frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{s}}{1 + \left\{41.6 + \frac{.00281}{s}\right\}\frac{n}{\sqrt{R}}}\right\}\sqrt{Rs}.$$

In using this method it is necessary to determine (1) the mean area of cross-section; (2) the slope of the surface of the stream; (3) data in regard to the roughness of the bed, from which to estimate the proper value of n. The channel conditions are so varying that it is possible to give only general rules for making slope measurements.

[a] See "The Flow of Water in Rivers and Other Channels" by Ganguillet and Kutter.

Examples for use of diagram.

n.	Find.	Procedure.
26 0015 5	c	Locate point where radial line $n = .026$ cuts slope curve $s = .00015$; join this point with point on hydraulic radius scale $R = 4.5$. This line cuts the vertical scale of coefficients in $c = 75$.
26 0015	R	Locate point $n = .026$, $s = .00015$; join this point with $c = 75$. The prolongation of this line cuts hydraulic radius scale in $R = 4.5$.
5 26	s	Join $R = 4.5$ with $c = 75$; prolong this line to cut radial line $n = .026$. This intersection falls on slope curve $s = .00015$.
5 0015	n	Join $R = 4.5$ with $c = 75$; prolong this line to cut slope curve $s = .00015$. This intersection falls on radial line $n = .026$.
26 0015 5	V	Find, as in Ex. 1, $c = 75$; draw line joining $R = 4.5$ with $s = .00015$ on vertical slope scale. A line through $c = 75$ parallel to this one cuts velocity scale in $V = 1.95$.
5 0015 95	n	Join $R = 4.5$ with $s = .00015$ on vertical slope scale. A line through $V = 1.95$ parallel to this one cuts scale of coefficients in $c = 75$. As in Ex. 4, find $n = .026$.
0015 26 95	R	Assume $R =$ say, 5; with $R = 5$, find, as in Ex. 5, $V = 2.10$—showing that assumed R is too large; try $R = 4$, and interpolate. That value of R which makes $V = 1.95$ is the true value.
5 95 26	s	Assume $s =$ say, .001; with $s = .001$, find, as in Ex. 5, $V = 5.0$—showing that assumed s is too large. Try $s = .0001$, and interpolate. That value of s which makes $V = 1.95$ is the true value.



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In making a measurement by this method the course or "run" must first be selected and measured, in which the slope and cross-section are reasonably uniform. The effective area of cross-section of this course is the mean of the cross-sections at its ends and at intermediate points.

The slope of the surface of the stream is obtained by simultaneous readings of gages placed at the ends of the run, as described on page 42. In case the course is not designed for continuous use for slope measurements, reference points from which the elevation of the water may be determined by a single vertical measurement may be used instead of gages.

In collecting data for determining the value of n it should be borne in mind that this factor includes not only the effect of roughness of bed but also that of all obstructions that may retard the water. In general n is larger for the overflow part or parts of the stream than for the channel proper; hence these parts should be treated separately in computing the discharge. For the higher stages of the stream n for the channel proper generally decreases as the stage increases. The engineer must rely largely on judgment and experience in this matter.

The results obtained by this method are in general only roughly approximate, owing to the difficulty in obtaining accurate measurements of slope and the other necessary data and the uncertainty of the value of n to be used in Kutter's formula.

But few definite data are available as to the values of the factor n. From the determinations that have been made in natural channels as given by Ganguillet and Kutter and in reports of Chief of Engineers, U. S. Army, and others, n has been found to vary from .020 to .025 for smooth, sandy, and fine gravel beds; from .030 to .035 for rough beds; and from .040 to .055 when the banks are overflowed, submerging fields covered with brush and débris.

For simplicity in computation tables giving the value of V and c for the various conditions have been published. Among these is Table XV, page 129. Diagrams (Pl. VI) are also used to advantage in this connection.

The slope method is commonly used for estimating flood discharge, often after the crest of the flood wave has passed, and when the only data available are the slope as shown by marks along the banks, and a knowledge of the general conditions. Under many other conditions, however, isolated approximate estimates of flow are made. Another important present use of Kutter's formula is in the design of canals, for which the slope must be determined in order that the channel may carry a certain quantity of water at a given velocity.

LOW-WATER MEASUREMENTS.

At many stations the velocity is small at low stages of the river. Where such conditions exist, it is advisable to find a section near by in which a meter measurement may be made by wading (Pl. V, A), and in which conditions of channel are suitable for a discharge measurement. Low-water measurements should preferably be made by the two-point or the three-point method.

Meters hung on a rod, as the Price acoustic meter, are best adapted for use in wading measurements. The small Price electric meter gives as good results, but is not so conveniently used. In making the measurements a tape line is stretched across the stream in order to mark the points of measurement. The engineer should stand below this tape line and preferably to one side of the meter, in order that he may not disturb the action of the meter. Three-eighths inch iron rods, 3 or 4 feet long and having a split in the top, may be conveniently used in these measurements for supporting the tape.

If the water is very shallow it is often necessary to confine the flow to a small channel in which there will be a measurable depth and velocity. If very accurate determinations are desired for small flows, sharp-crested weirs should be installed.

HIGH-WATER MEASUREMENTS.

Measurements made at high stages generally consist of observations of surface velocity only. Areas must be computed from a standard section or from soundings made at a lower stage. Under these conditions extra precautions must be taken to secure data from which a reliable estimate of the flow can be made. On account of the great velocity and the presence of drift and cakes of ice, it is often practically impossible to make a good meter measurement. Meter observations of 10 to 30 seconds may, however, frequently be obtained when there is considerable drift, but great care must be exercised that the meter is not damaged. If a weight of more than 30 pounds is required to submerge the meter, cotton or manila line must be used in conjunction with the insulated cable for supporting the meter. Extra heavy and large tails with both horizontal and vertical vanes must also be attached to the weights in order to keep the meter headed properly into the current.

If there is much drift, however, it is generally advisable to use the float method, the drift serving as floats. In this event a base line is

laid off on the shore, range poles are set on one or both banks, and the surface velocities are obtained by observing the time required by the drift to pass over this measured course in various parts of the channel. In other instances, it may be practicable to determine the slope of the river for a considerable distance. If a level is not at hand, marks may be made and the slope determined at some future visit. When possible, two of these methods may be used and a check thus obtained upon the work.

MEASUREMENT OF ICE-COVERED STREAMS.[a]

Measurements of discharge of a stream with an ice cover may be made either by a weir or current meter. More than ordinary care must be exercised in collecting the records, however, which include, in addition to those made during the open-water season, full notes of ice conditions.

At weir stations the computed discharge may be as accurately made in winter as during the open-water season if the crest of the weir is kept free from ice and if the ice cover above is not too near—say within 20 feet of the crest. In many instances, however, the crest of the weir becomes so covered and obstructed either by the formation or lodging of ice that computations of flow are much in error.

Current-meter measurements are made by soundings and observations of velocity in holes cut through the ice, and the individual observations may be as accurately made as in open water.

The table on pages 66–67, giving a summary of the results of all vertical velocity-curves available, shows that there are two points in which the thread of mean velocity occurs under ice. These points are at about .1 and .71 depth below the bottom of the ice, varying between 0 and, .22 for the upper and .63 and .79 for the lower. It is thus seen that they lie very nearly at .2 and .8 of the depth. As seen in the table the .2 and .8 depth method gives the mean velocity within a small percentage of error, and in such measurements this method should be used in case it is impossible to make the whole measurement by vertical velocity-curves (fig. 13). If many observations are to be made, a canvas shelter which may be moved along on the ice from hole to hole will facilitate the work on account of the greater comfort of the workers.

The record of stage may be kept on any kind of a gage and should include observations of the elevation of the water surface as it rises in a hole cut through the ice, the elevations of the surface and bottom of the ice, and the thickness of the ice. All of these records are necessary because the ice rests differently on the water under different conditions.

[a] See Water-Supply Paper No. 187, U. S. Geol. Survey.

Summary of vertical velocity curves under ice cover.

SMOOTH ICE COVER.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Depth of threads of velocity.			Coefficient to reduce to mean velocity.			Bed of stream.
							Upper mean.	Lower mean.	Maximum.	Maximum.	0.5 depth.	$\frac{0.2+0.8 \text{ depth}}{2}$	
			Feet.	*Feet.*	*Feet.*	*Ft. per sec.*							
Catskill	South Cairo, N. Y.	4	2.82	2.7	0.98	0.44	0.06	0.70	0.31	0.80	0.86	0.98	Gravel.
Chemung	Chemung, N. Y.	4	2.85	3.7	.51	.64	.16	.79	.46	.86	.85	1.02	Coarse gravel.
Connecticut	Orford, N. H.	18	4.16	4.0	1.97	1.04	.11	.69	.37	.85	.88	1.00	Gravel and sand.
		12	± 4.1	4.1	1.91	.92	.07	.63	.25	.83	.92	.99	
		6	5.6	7.3	....	1.80	.12	.73	.37	.87	.89	.99	
Esopus	Kingston, N. Y.	2	5.28	5.8	.10	.94	.10	.74	.52	.87	.87	1.02	Sand and gravel.
		8	5.60	3.4	.56	.80	.03	.63	.24	.76	.85	1.01	
		12	6.6	4.9	.10	1.19	.07	.69	.29	.82	.88	.98	
		8	7.0	5.0	.50	1.26	.08	.67	.36	.82	.86	.98	
Fish	Wallagrass, Me.	8	3.91	2.5	1.40	.90	.07	.70	.36	.88	.89	1.00	Gravel.
		8	5.00	3.5	1.45	1.25	.14	.77	.43	.84	.86	.98	
Genesee	Mount Morris (Jones Bridge), N.Y.	20	5.9	3.8	.54	1.62	.07	.67	.34	.80	.85	1.01	Coarse gravel.
Genesee	Rochester, N. Y.	11	1.42	6.6	.45	.45	.09	.72	.44	.84	.86	.99	Do.
		19	1.55	3.8	.43	.89	[a].01	.70	.33	.82	.87	1.00	
Kennebec	North Anson, Me.	4	3.40	2.7	....	2.27	....	.63	.19	.82	.92	1.00	Gravel.
		12	3.48	2.6	1.30	1.17	.04	.66	.30	.87	.91	.02	
		15	3.55	1.6	2.15	1.87	.18	.77	.45	.81	.82	1.01	
		19	4.17	2.5	1.91	1.30	.07	.69	.34	.86	.90	1.00	
		10	4.70	3.0	2.04	1.32	.10	.71	.37	.87	.89	1.00	
		9	4.77	2.9	2.34	1.33	08	.70[a]	.32	.87	.90	.98	
		23	5.30	3.2	2.00	1.61	.12	.70	.42	.83	.86	1.01	
Mohawk	Utica, N. Y.	13	[b] 16.2	10.1	.50	1.00	.21	.78	.47	.82	.84	1.04	Earth, very smooth.
Raquette	Massena Springs, N. Y.	17	9.6	9.2	1.60	3.24	.17	.74	.43	.90	.88	1.00	Coarse gravel.
Rondout Creek	Rosendale, N. Y.	3	5.15	6.7	.75	.55	([c])	.65	.24	.80	.89	1.00	Rock.
Wallkill	Newpaltz, N. Y.	11	7.3	6.0	±1.2	.83	.11	.70	.40	.83	.87	1.01	Clay and coarse gravel.
		4	7.78	7.9	1.05	1.29	.22	.76	.46	.82	.90	1.02	
		14	8.95	7.2	1.00	1.57	.10	.70	.36	.84	.89	1.00	
		13	11.2	9.5	.85	2.35	.11	.71	.36	.85	.89	1.00	
		10	17.33	18.2	.92	3.54	.16	.79	.43	.89	.91	.90	

[a] Upper thread of mean velocity at 4 curves only.

[b] Below reference point.

[c] Upper thread of mean velocity for 1 curve only.

Summary of vertical velocity curves under ice cover—Continued.

SMOOTH ICE COVER—Continued.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Depth of threads of velocity. Upper mean.	Depth of threads of velocity. Lower mean.	Depth of threads of velocity. Maximum.	Coefficient to reduce to mean velocity. Maximum.	Coefficient to reduce to mean velocity. 0.5 depth.	Coefficient to reduce to mean velocity. $\frac{0.2+0.8 \text{ depth}}{2}$	Bed of stream.
			Feet.	*Feet.*	*Feet.*	*Ft. per sec.*							
West Canada Creek	Twin Rock Bridge, N. Y.	9	3.8	2.5	1.50	1.82	0.04	0.66	0.34	0.86	0.91	1.02	Coarse gravel.
Winooski	Richmond, Vt.	26	5.5	1.6	2.10	2.12	.12	.72	.40	.80	.83	1.02	Gravel.
Mean of 352 curves							.10	.71	.37	.839	.878	1.002	
Highest							.22	.79	.52	.90	.92	1.04	
Lowest							.00	.63	.19	.76	.82	.98	

ROUGH ICE COVER.

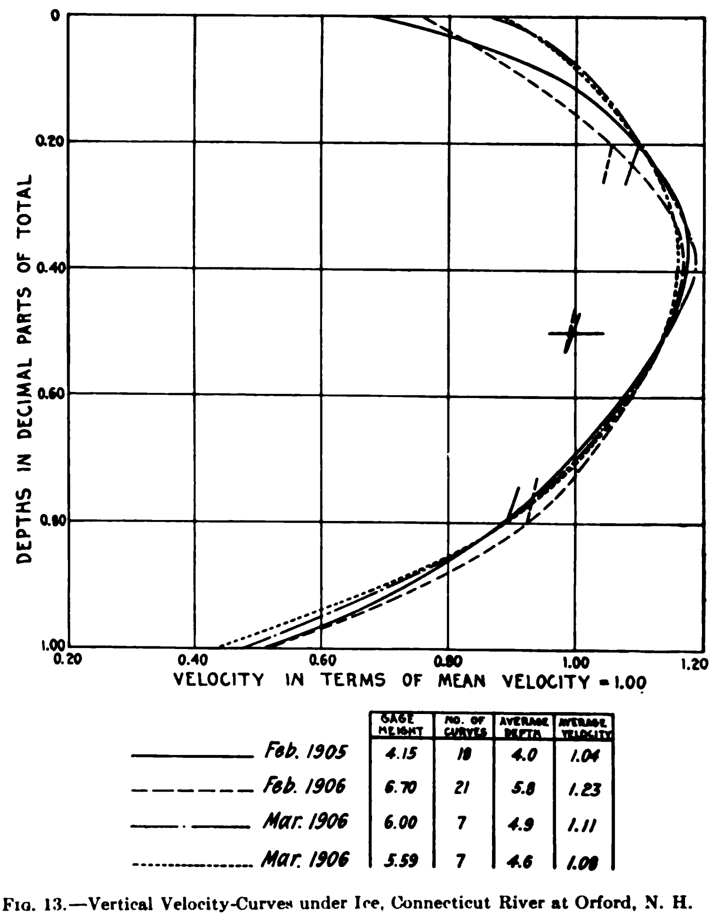
River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Upper mean.	Lower mean.	Maximum.	Maximum.	0.5 depth.	0.2+0.8 depth / 2	Bed of stream.
Connecticut	Orford, N. H.	7	5.59	4.6	1.71	1.08	0.08	0.70	0.37	0.86	0.88	1.00	Gravel and sand.
		7	6.00	4.9	1.69	1.11	.09	.70	.39	.84	.88	1.00	
		21	6.70	5.8	1.66	1.23	.14	.74	.42	.86	.88	1.00	
Des Moines	Keosauqua, Iowa	5	2.45	4.8	.46	1.58	.13	.73	.40	.86	.88	1.00	Gravel.
Maumee	Sherwood, Ohio	11	3.34	2.5	.58	.78	.11	.68	.34	.92	.95	1.01	Do.
Mean of 51 curves							.11	.71	.38	.868	.894	1.002	
Highest							.14	.74	.42	.92	.95	1.01	
Lowest							.08	.68	.34	.84	.88	1.00	

VERY ROUGH ICE COVER, BROKEN AND TILTED.

River.	Station.	Number of curves.	Gage height to water surface.	Depth under ice.	Ice thickness.	Mean velocity.	Upper mean.	Lower mean.	Maximum.	Maximum.	0.5 depth.	0.2+0.8 depth / 2	Bed of stream.
Rondout Creek	Rosendale, N. Y.	4	± 7.0	5.3	0.45	0.74	0.27	+[a]0.88	0.56	0.80	0.85	1.04	Rock.
Wallkill	Newpaltz, N. Y.	4	13.7	14.6	1.40	2.98	.27	.88	.56	.84	.85	1.00	Clay and coarse gravel.
Winooski	Richmond, Vt.	5	5.62	6.2	±2.5	1.99	.22	.81	.52	.74	.76	1.02	Sand and gravel.
Mean of 13 curves				8.7	1.45	1.90	.25	.86	.55	.79	.82	1.02	

[a] No lower mean thread of velocity for 1 curve.

If normal, it should stand about one-eighth of its thickness above the water, but on a falling stream may stand farther above, and in extreme cases, on a small stream, it may bridge it from bank to bank without touching the water, or on a rising stream the water may rise to the surface or even above the ice if the ice is frozen fast to the banks. The



	Gage Height	No. of Curves	Average Depth	Average Velocity
Feb. 1905	4.15	18	4.0	1.04
Feb. 1906	6.70	21	5.8	1.23
Mar. 1906	6.00	7	4.9	1.11
Mar. 1906	5.59	7	4.6	1.08

FIG. 13.—Vertical Velocity-Curves under Ice, Connecticut River at Orford, N. H.

record of stage is therefore much more difficult to obtain than in the open season. Since the winter discharge is largely maintained from ground water, the range of stage is small and changes occur slowly. It is consequently sufficient for estimating the daily discharge to have the gage cut out once or twice a week and records made as indicated above.

Ice conditions have a wide range, from a narrow fringe along the shore to a complete smooth ice cover and from the latter to broken ice, which may be jammed many feet in height and extend in places to the bed of the river. Changes in these conditions sometimes occur with considerable rapidity.

The record of stage under ice conditions is of no value unless the stage is an index of discharge, which is not the case if the ice breaks and jams, or if anchor ice forms or collects in the vicinity of the gage. All ice conditions must therefore be carefully watched and recorded, but even with the best observations the application of a rating curve to a record of stage will be attended with much greater error than in open-water conditions.

Ice conditions may be divided into six classes, which should be considered in selecting current-meter gaging stations or in collecting records at such stations:

1. Smooth, permanent ice cover.
2. Tendency for anchor or needle ice to accumulate underneath ice cover.
3. Unstable ice cover due to—
   (*a*) Effect of warmer inflow from lakes or tributary streams.
   (*b*) Effect of inflow of ground water.
   (*c*) Effect of warm current due to artificial causes, such as factory waste, etc.
   (*d*) Concentrated quick water and wearing away due to friction.
   (*e*) Considerable fluctuation in stage occasioned by winter freshets.
4. Rough ice cover and piling up of ice due to quick water and rough bed.
5. Tendency for ice jams to occur, with consequent backwater, etc.
6. Open water altogether, or water frozen over thinly for short times, owing to—
   (*a*) Extremes of conditions as noted under 3.
   (*b*) High temperature, mainly in the southern portion of the area subject to ice conditions in winter.

The first class is the only one of the six at which it is possible to make estimates of discharge from records of gage height and occasional measurements of discharge.

MEASUREMENTS IN ARTIFICIAL CHANNELS.

The method used in the measurement of flow in artificial channels should be studied with special care, as the laws of flow in open channels

do not always hold in artificial channels, because of disturbing influences, such as undercurrents caused by intakes and outlets, and rapid variations in the water level of the canal.

For uniform cross-sections tube floats can be used to good advantage. If measurements are made by current meters the vertical velocity-curve or the integration method should be used, and investigations should be made to determine whether the velocity in different portions of the cross-section is rapidly changing.

MAINTENANCE OF STATIONS.

In addition to the observations which must be made by the engineer at the time of each discharge measurement, as outlined above, the datum of the gage should be checked at least once in each year by comparison with a permanent bench mark and a careful cross-section developed. If it is necessary or desirable to replace a gage, the new one should be made to read from the same datum as the old one, unless there is some excellent reason to the contrary. In any event, a complete record should be made of what has been done, special care being taken that any changes in the datum shall be clearly recorded.

Either temporary or permanent changes in channel conditions which affect the rating of the station should be noted and recorded in as great detail as possible. Such conditions include changes in channel in the vicinity of the station, the building of dams below, the formation of jams of logs or drift, and the extent, character, and thickness of ice. Gage readers should receive written instructions in regard to reports to be made concerning such conditions; otherwise no record may be made of the time or extent of such changes.

If the engineer can reach a gaging station in flood it will generally be economy for him to remain and make measurements of discharge at each foot of stage as the river descends. In this way the station may be practically rated for all except low stages in a single freshet, and with a minimum expenditure.

CHAPTER V.

WEIR STATIONS.

In a broad sense a weir is any artificial structure placed in a stream for the purpose of raising the surface of the water. A weir for measuring discharge must have a well-defined form and a reasonably level crest of permanent shape and elevation, and must not allow a large percentage of the water to pass through, beneath, or around it.

Weirs may be used for measuring the quantity of water in streams because water flows over them in accordance with known, definite laws. They become available for such measurement by the use of formulas in which the quantity of discharge is expressed in terms of the dimensions of the weir and the head of water on its crest, and by coefficients which have been determined by experiments.

Weirs may be divided into two classes—(1) sharp-crested, or standard weirs, and (2) broad-crested weirs or dams, the distinction depending on whether the water in passing over them comes in contact with the crest on a line or a surface. Either of these two classes may be submerged or may have a free overfall—that is, the elevation of the water on the downstream side of the weir may be above or below its crest. Weirs of either of these classes may be vertical or inclined. Usually measurements of flow are made only at vertical weirs having a free overfall, and the following discussion is limited to that class.

In considering the establishment of a weir station, choice must generally be made between a velocity-area station, the use of an existing dam, or the construction of a sharp-crested weir to be used exclusively for obtaining a record of flow. The choice between these types of stations will be governed largely by conditions relating to the cost and accuracy of the records.

SHARP-CRESTED WEIRS.

Sharp-crested weirs used under heads of not to exceed 5 feet offer the best facilities known for determining the flow of streams whose discharges are too great to be weighed or measured in a calibrated tank.

The coefficients for use in formulas for such weirs have been carefully determined for heads under 5 feet, and have a small range in value.

The use of sharp-crested weirs is generally limited by their cost to comparatively small streams or to streams of which very accurate records of flow are desired. They are most commonly employed to divide water among several users, especially for irrigation, and the principal requisite for their location is a site at which the weir can be economically constructed so that there will be no percolation or leakage.

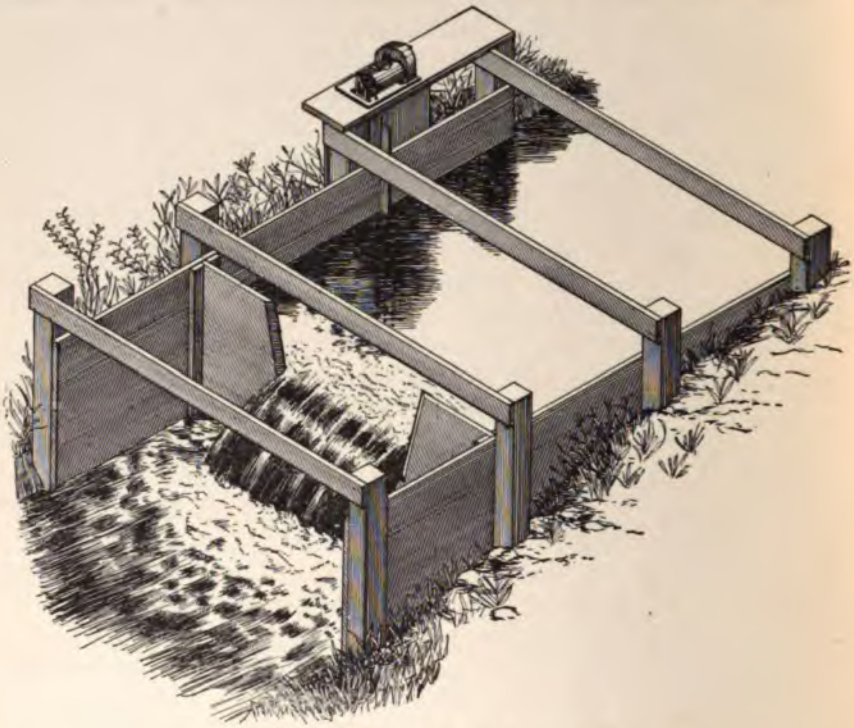


FIG. 14.—Cippoletti Weir, with Water Register in Place.
(Mead, Bulletin 86, Experiment Stations, Department of Agriculture.)

Sharp-crested weirs may be either rectangular or trapezoidal in form and must have a crest of such dimensions and height that the water will have free fall over it with provision for the admission of air under the overfalling water.

As commonly arranged, the weir projects sharply from both sides and the bottom into the channel conducting the water, thus making the dimensions of the cross-section over the weir less than those of the

channel of approach. This reduction in the cross-section of the channel causes a contraction of the water at the bottom and the ends as it passes over the weir. If both end and bottom contractions exist the weir is called a *contracted* weir. This contraction may be prevented by arranging the channel of approach so that the water is guided both on the bottom and ends directly to the crest of the weir, making what is called a *suppressed* weir. In many weirs the end contractions only are suppressed, when the weirs are said to be partially contracted.

End contractions cause a virtual decrease in the length of crest of the weir. For rectangular weirs this effect is provided for in the formulas. The Cippoletti weir (fig. 14), which is the most common form of trapezoidal weir, is constructed with the outward slope of each end 1 horizontal to 4 vertical. This causes an increase in effective length as the head increases, thus very nearly compensating for end contraction.

In sharp-crested weirs the channel of approach, fore bay, hydrant, or stilling box from which the water flows over the weir must either be sufficiently large to eliminate velocity of approach to the weir, or a correction must be made for such effect in the computations. The structure will therefore vary in size and arrangement for the accommodation of different quantities of water.

BROAD-CRESTED WEIRS.[a]

Weir stations on large streams will usually be located at existing dams (Pl. VII, B) constructed for purposes of power or navigation, and selection must be made between several available dams or between a dam and a velocity-area station. In either case the advantages and disadvantages of each locality must be carefully considered, as the value of the resulting record of discharge will depend largely upon the possibilities of the station. As compared with velocity-area stations dams may have the advantage of continuity of record through the period of ice but the disadvantage of uncertainty of coefficients to be used in the weir formulas and complications due to diversion and use of water.

In investigating the availability of a dam for gaging purposes, observations must be made concerning certain conditions which are necessary to insure good records. These conditions may be divided into two classes—(1) those relating to the physical characteristics of the dam itself, and (2) those relating to the diversion and use of water around and through the dam.

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[a] Stations at broad-crested weirs are fully discussed in U. S. Geol. Survey Water-Supply Papers Nos. 200 and 180.

The physical requirements are as follows: (*a*) Height of dam such that backwater will not interfere with free fall over it; (*b*) absence of leaks of appreciable magnitude; (*c*) topography or abutments which confine the flow over the dam at high stages; (*d*) level crests which are kept free from obstructions caused by floating logs or ice; (*e*) crests of a type for which the coefficients to be used in $Q = clH^{\frac{3}{2}}$ or some similar standard weir formula are known; (*f*) either no flash boards or exceptional care in reducing leakage through them and in recording their condition.

Preferably there should be no diversion of water through or around the dam. Generally, however, part or all of the water is diverted for uses of power or navigation. Such water must be measured and added to that passing over the dam. To insure accuracy in estimates the water diverted must be reasonably constant in quantity, and so utilized that it can be measured either by a weir, a current meter, or through a simple system of water wheels which are of standard make or have been rated as water meters under working conditions and so installed that the gage openings, heads under which they work, and their angular velocities may be accurately observed.

The combination of physical conditions and uses of the water should be such that the estimates of flow will not involve, for a critical stage of considerable duration, the use of a head on a broad-crested dam of less than 6 inches. Moreover, when all other conditions are good, a careful observer is still essential in order to obtain reliable results.

The field work for the establishment of a station at a dam must be sufficient to provide for obtaining the records of gage height, and must also include the surveys and the collection of information which will make possible the correct interpretation and application of these records in the computation of discharge. It must consist, therefore, of the establishment of a gage for determining the head on the dam, and, if water is diverted through a head race and used through wheels or wasted through gates or over weirs, the establishment of sufficient other gages for determining the effective head on such turbines, gates, or weirs. Provision must be made for the systematic reading of these gages as well as for recording the conditions of wheel-gate openings, speed of wheels, elevations of crests of adjustable waste weirs, openings of waste gates, and elevation and conditions of flash boards. The gages must each be referenced by a convenient bench mark and all connected by a line of levels. An instrumental survey of the dam must be made to determine the length, profile, and cross-section of the crest. Cross-

A. TYPICAL BOAT STATION.

B. WEIR STATION, QUABOAG RIVER, WEST WARREN, MASS.

sections of the channel of approach to the dam should also be measured in order to estimate the velocity of approach. Usually special forms for records and computations must be prepared for each such station.

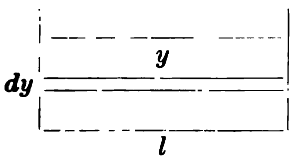
WEIR FORMULAS[a]

The discharge over a weir is the product of the area of effective cross-section of the vein of water passing over it, the mean velocity in this area, and a coefficient determined experimentally, which varies with the form of the weir. The area of cross-section is determined approximately by the length of the crest and the head or the depth of water on it. The velocity is determined approximately by the head. These two quantities, length of crest and head, together with the coefficient, are therefore factors entering all weir formulas. They must, however, be modified for differences in forms of weirs, conditions of contraction, and velocity of approach.

The effects of end contraction and velocity of approach are allowed for in the formula by modifying the length of the crest and head respectively, or in the coefficient. The coefficient to be used in any instance must have been determined for that particular formula.

FUNDAMENTAL FORMULAS.

The fundamental formula for rectangular weirs may be derived by the calculus as follows:



$$Q = c\int_0^H \sqrt{2gy}\; l\,dy = \tfrac{2}{3}\, c\, l \sqrt{2g}\, H^{\frac{3}{2}} \qquad (1)$$

in which l represents the length of the weir; H, the head of water on the weir; y, the head on any horizontal strip of differential width, dy; g, the acceleration of gravity; and c, coefficient that must be determined experimentally and that varies with different conditions of crest, channel of approach, etc. In the integral expression, $\sqrt{2gy}$ is the theoretical velocity of the water in the strip whose area is ldy. The integration between the limits 0 and H of the products of the infinitesimal areas by the velocities through them gives the total discharge, Q.

Modifications are made necessary, as previously explained, by reason

[a] See Hydraulics, by Hamilton Smith, jr.

of velocity of approach, variations in contraction of the water as it passes the weir, or variations in form of weir. If the end contraction is perfect, it causes at each end of the weir a shortening of the effective length by approximately .1 H.

If allowance is made for such end contraction, formula (1) becomes

$$Q = \tfrac{2}{3}\, c(l - .2H)\sqrt{2g}\, H^{\frac{3}{2}} \qquad (2)$$

The same results are also accomplished in a different way by modifying properly the coefficients used in formula (1).

The velocity of approach " V " causes a virtual increase in head. The magnitude of such increase is the head corresponding to that velocity, is represented by h, and equals $\frac{V^2}{2g}$. Such velocity of approach may be obtained approximately as the quotient by dividing the discharge by the area of cross-section of the channel of approach. The result so obtained should, however, be multiplied by a coefficient greater than unity (usually assumed to be between 1 and 1.5). Since the amount of the discharge is the quantity to be determined, the approximate value of V must be found from an approximate determination of Q by an application of the weir formula, neglecting the velocity of approach.

The correction for velocity of approach may be effected by adding h directly to the measured head in formula (1), as follows:

$$Q = \tfrac{2}{3}\, cl\sqrt{2g}\,(H + h)^{\frac{3}{2}} \qquad (3)$$

or the correction may be applied before integration as follows:

$$Q = c\int_0^H \sqrt{2g\,(y + h)}\; l\,dy = \tfrac{2}{3}\, cl\sqrt{2g}\left[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right] \qquad (4)$$

Formulas (3) and (4) are both in common use.

The formulas already explained—(1), (2), (3) and (4)—serve as bases for the formulas of all free overfall rectangular weirs, whether sharp or broad-crested. Values of c have been determined for use in each of these formulas for various types of weirs. Many sets of coefficients are therefore available, but each is applicable only to its formula.

RECTANGULAR WEIRS.

The formulas shown above, or slight modifications of them, are in general use for rectangular weirs. Of the modifications in common use, the *Francis formula* (5) is the simplest in form and application.

Francis determined that for a suppressed weir, without velocity of approach, c had an average value of 0.62. The product of the three constants of formula (1), 0.62, $\frac{2}{3}$, and $\sqrt{2g}$ = 3.33, thus making

$$Q = 3.33\ lH^{\frac{3}{2}} \tag{5}$$

The discharge per foot of length as determined by this formula is given in Table II, pages 116–117.

When modified to allow for end contractions and velocity of approach, formula (5) becomes

$$Q = 3.33\ (l-.2H)\ \left[(H + h^{\frac{3}{2}}) - h^{\frac{3}{2}}\right] \tag{6}$$

If there is no velocity of approach formula (6) becomes

$$Q = 3.33\ (l-.2H)\ H^{\frac{3}{2}} \tag{7}$$

Table I, pages 114–115, shows the discharge determined by formula (7).

In the use of formulas (5), (6), and (7), the dimensions must always be expressed in feet, because that unit has been introduced in the value of g, which appears in the coefficient.

Other formulas in common use were devised by *Bazin*, among which is the following, which gives the discharges for a sharp-crested weir without end contractions:

$$Q = \left(0.405 + \frac{.0984}{H}\right)\left(1 + 0.55\frac{H^2}{(p + H)^2}\right) lH\sqrt{2gH} \tag{8}$$

in which H = observed head in feet; p = height of weir in feet; l = length of crest in feet; Q = discharge in second-feet.

Table IV, pages 120–122, shows the discharge as computed from formula (8).

TRAPEZOIDAL WEIR.

The trapezoidal weir is unimportant, except the Cippoletti weir (fig. 14), in which the outward slope of the ends (p. 73) counteracts the decrease in length due to end contractions. Ordinary formulas for suppressed weirs are therefore approximately applicable to it. Table II may be used for the Cippoletti weir with an error of about 1 per cent, giving results too small by that amount. Special determinations of coefficients for this weir have, however, been made and the resulting formula for discharge without velocity of approach is

$$Q = 3.3\tfrac{2}{3}\ lH^{\frac{3}{2}} \tag{9}$$

BROAD-CRESTED WEIRS.

Coefficients for several types of broad-crested weirs have been determined by Bazin, in France, and under the direction of Prof. Gardner S. Williams at the Cornell University Testing Laboratory, for the United States Deep Waterways Commission, by Mr. John R. Freeman and by members of the United States Geological Survey. The results of all of these experiments have been brought together by Mr. R. E. Horton in Water-Supply Paper No. 200, United States Geological Survey.

In this paper it is shown that within certain limits of head the discharge over several types of broad-crested weirs may be found by the use of formula

$$Q = 2.64\, lH^{\frac{3}{2}} \tag{10}$$

This formula is applicable to broad-crested weirs of any width of cross-section exceeding 2 feet within such limit of head that the nappe does not adhere to the downstream face of the weir for low heads nor tend to become detached from the crest with greater heads. If the latter condition exists, the coefficient increases to a limit near the value which applies for a thin-edged weir, a point being finally reached where the nappe breaks entirely free from the broad crest and discharges in the same manner as for a thin-edged weir. Formula (10) may be applied safely to any weir having a crest width exceeding 2 feet and with heads from 0.5 foot to 1.5 or 2 times the breadth of weir crest. Table III, pages 118–119, shows the discharge as determined by this formula.

From the experiments mentioned above Mr. E. C. Murphy has developed multipliers to be used in connection with Bazin's formula for discharge over a sharp-crested weir to find the discharge over a broad-crested weir. Tables V, VI, and VII, pages 123–125, and fig. 24 show these multipliers and the forms of weirs to which they pertain. These tables are to be used in connection with Table IV, which has been made the basis in their computation.

COMPUTATIONS.

In the computation of discharge over a weir, whether sharp or broad-crested, a rating table is first prepared which gives the discharge for the various heads occurring during the period of observation. This rating table is computed by substituting values of head, dimensions of weir, and coefficients depending upon the type of weir under consideration in the formula applicable to such weir.

Many dams, unless built of solid masonry, have irregular crests due to unequal settlement. Such a dam must be divided into parts, each of which has a uniform elevation of crest, the formula applied to each part independently, and the results combined to form the rating table.

If any fixed condition of flash boards or other modification of the crest of the dam exists for a considerable period of time, a similar rating table should be made also for such condition. In the same way it will be found to be economical to compute rating tables for any fixed waste weirs in the head-race and for the usual condition of waste gates, wheel gates, etc., which are sufficiently constant to warrant such computations.

With rating tables at hand as above described, the computations of daily discharge are made by entering each rating table for the partial discharge through or over the structure for which that table has been made. The partial discharges so obtained are summed to give the total rate of flow. Discharges at stages and for conditions which are not covered by the rating tables must be computed independently.

These rating tables are as a rule instrumental in saving time in the computations, but their principal value arises from the fact that errors are much less likely to appear in the results than if each discharge is computed independently from a formula. For the same reason tables are more satisfactory than diagrams.

CHAPTER VI.

DISCUSSION AND USE OF DATA.

COMPUTATION OF DAILY DISCHARGE.

The results of the occasional discharge measurements, records of gage heights, and descriptions of conditions at a gaging station are utilized to compute the daily discharge of the stream. From the daily discharge are derived various other values relative to discharge and run-off, which vary with the use to be made of the data. They include mean, maximum, and minimum discharge and run-off per week, month, or season; the duration of various stages and their corresponding discharges; and the discharge in various units from the drainage areas under consideration.

The daily discharge of a stream may be computed in various ways, the method to be chosen depending upon whether the measurements of discharge are made by the weir or the velocity-area method. Such computations for weir stations are described in Chapter V. The methods used for the velocity-area stations depend on the degree of permanency of the conditions that affect the flow and are discussed in the following pages.

Like most so-called natural laws, those pertaining to the flow of streams apply only under certain conditions and vary within certain limits. In general, it may be said that the discharge of a stream is a function of the gage height and that for the same stage there will always be the same discharge. This principle of stream flow holds approximately true so long as the conditions controlling the flow in the vicinity of the gaging station remain reasonably constant, and upon this principle are based the methods of computing the daily discharge of streams.

GAGING STATIONS WITH PERMANENT CONTROL.

For stations on streams with permanent beds it is possible to prepare, from the data collected, station rating tables, each of which gives for its station the discharge corresponding to any stage of the stream, and

which, when applied to the daily gage heights, gives the daily discharge. The basis for a station rating table is a rating curve which shows graphically the discharge corresponding to any stage of the stream and is usually constructed by plotting the results of the various discharge measurements with gage heights as ordinates and discharges as abscissas. These points define the curve, which is then drawn by use of French curves.

If accurate and well-distributed discharge measurements covering the range of stage are available, the station rating curve can be readily constructed. Frequently, however, the measurements are more or less discordant and do not cover all stages. As a result special studies are necessary to determine the relative accuracy of the measurements and the location of the curve.

Since the discharge is the product of two factors, the area and mean velocity, any change in either factor will produce a corresponding change in the discharge. The curves of area and mean velocity furnish, therefore, valuable assistance in studying the accuracy of the measurements and in determining the true location of the rating curve. These curves are defined by plotting gage heights as ordinates, and area and mean velocity, respectively, as abscissas.

AREA CURVE.

The curve of area shows the relation between the gage height and the area of the cross-section of the stream. This area must include both moving and still water in order to be of use for comparison. Two factors, the width and depth, or gage height, govern the form and position of the curve, which is normally concave to the X axis but may, under special conditions, be straight. For ordinary conditions, where the width increases with the stage, the curve may be assumed to be a series of parabolic arcs whose parameters vary with the slope of the banks. If the banks are vertical the increment is constant and the curve becomes a straight line. It is never concave to the Y axis unless the unusual condition of overhanging banks exists.

The area curve can always be definitely located from a careful series of soundings, which should be taken at low water, during the period over which the discharge curve is to apply, and be developed to high water by use of a level. The curve can be constructed easily, and generally with sufficient accuracy, by determining the area only at those gage heights at which the slopes of the banks change. If extreme accuracy is desired the area should be computed for each half-foot of

gage height. High-water soundings and those made in deep streams in which the velocity is great are liable to large errors, and areas computed from them should be carefully scrutinized. Such soundings have been more prolific of sources of error in discharge measurements than all other factors combined.

Since for an infinitesimal change in stage the increase in area equals the product of the width at that stage by the difference in gage height, it follows that the width equals the quotient of the increase in area divided by the difference in gage heights, which ratio is the tangent of the angle that the area curve makes at that stage with the vertical; therefore the direction of the area curve for any stage is determined by

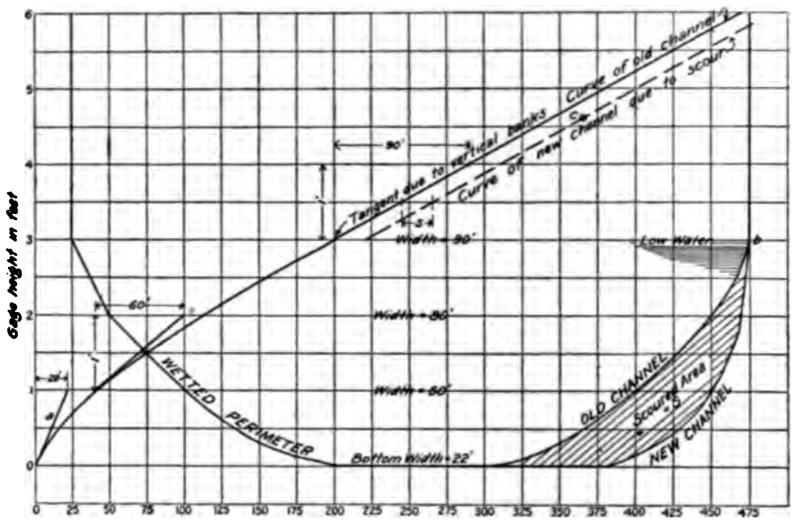


Fig. 15.—Typical Area Curves, Illustrating Their Form.

plotting from the vertical the angle whose tangent is the width at that stage. As most area curves are distorted by magnifying the vertical scale, the above principle is utilized by laying off unity on the vertical or gage-height axis to the scale of gage heights, and the width on the horizontal or area axis to the scale of area (fig. 15).

There follow from the above property the following useful characteristics of such curves when referred to origins of coordinates at the elevation of the lowest point in the cross-section: (*a*) For all sections except those with flat bottoms the area curve becomes tangent to the Y axis at the origin; (*b*) if the bottom is flat the curve cuts the Y axis at the origin at an angle whose tangent is the width of the bottom

(*a*, fig. 15); (*c*) if the banks are vertical the increment is constant and the curve proceeds in a straight line (fig. 15); (*d*) the area curve is permanent in curvature for all gage heights above the plane below which all shifts occur (fig. 15).

The accuracy of the areas as measured at the time of discharge measurement may be quickly tested by plotting them and drawing through each a straight line whose direction is tangent to the curve at that gage height and is determined by the width of the stream, as explained above. The curve should then be drawn from mean low water and kept parallel to the tangents at each point. Errors and discrepancies are at once

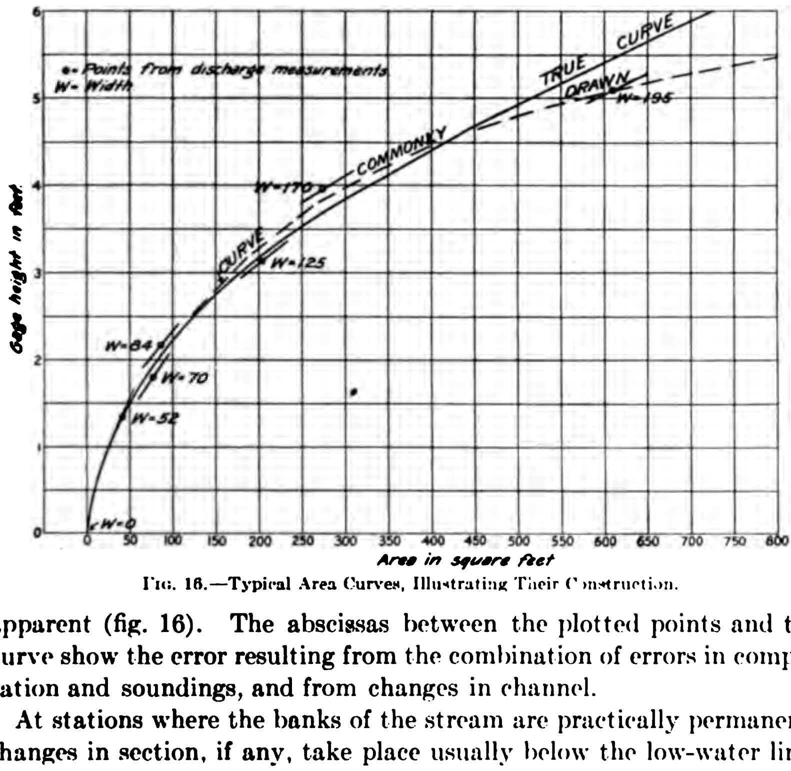


FIG. 16.—Typical Area Curves, Illustrating Their Construction.

apparent (fig. 16). The abscissas between the plotted points and the curve show the error resulting from the combination of errors in computation and soundings, and from changes in channel.

At stations where the banks of the stream are practically permanent, changes in section, if any, take place usually below the low-water line. If the area of such a section changes, the part of the curve above low water, which has been accurately constructed, may be shifted a proper distance horizontally to the right or left and be made to show accurately the areas of the new cross-section (fig. 15). The constant abscissa length between the old and new position of the curve is the algebraic sum of the changes in the area of the section, + for gain in area by

scour and — for loss in area by fill. A single determination of area at any gage height above low water therefore determines the new position of the curve, *c* (fig. 15).

MEAN VELOCITY CURVE.

As stated in Chapter IV, the mean velocity of the stream is the average rate of motion of all the filaments of water of the cross-section and depends principally upon (1) the surface slope of the stream, (2) the roughness of the bed, and (3) the hydraulic radius, and has been expressed in the Chezy formula as $V = c\sqrt{Rs}$, in which the coefficient c has been expressed by Kutter in terms of s, R, and n.

Since slope is the most important factor affecting velocity, when the rate of change in the slope is rapid the velocity tends to follow such change. When the slope becomes constant, changes in the velocity are controlled by the other two factors, the hydraulic radius and the coefficient of roughness.

The curve of mean velocity shows the relation between the gage height and the mean velocity of the current in the measured section. It is located by means of points which are determined by plotting the gage heights and corresponding mean velocities as coordinates. If sufficient measurements have been made to define the curve throughout the range of stage, no further investigation of its properties will be necessary. It frequently happens, however, that the curve must be constructed from limited or discordant values of velocity. To do this intelligently and satisfactorily a knowledge of the properties of the curve under various conditions of flow is essential, and in such cases it is advisable to develop the curves of R and s.

For usual conditions of natural flow in which the stage of no flow is the lowest point in the measured section, the mean velocity curve has approximately the form of a parabola with axis vertical and origin at or below the bed. It approaches a straight line, however, for the higher stages.

When the gaging section is in a stretch of the stream where zero flow occurs with ponded water at the section of the gage, the mean velocity curve will reverse at low stages and approach zero convex to the gage axis. The degree of curvature and the point at which the curve reverses are apparently governed chiefly by the amount of ponded water at the gage, the roughness of the bed, the form of the controlling bar, and other channel conditions. If the stream is small and shallow the change in direction is more abrupt. This peculiar reversal is probably

due to the rapid rate of change of the slope at extreme low flow. At zero flow the slope is of course zero. The least flow causes a slope of the surface and this slope increases with the stage, up to a certain point.

Three methods of extending the mean velocity curve from medium stages to high water have been employed: (1) Extend the curve as a tangent from the last observed value; (2) extend the curve as a hyperbola, i. e., approaching the straight line as its asymptote; (3) assume the slope constant or increasing slightly over the intermediate stages and compute the value of the velocity from the formula $V = c\sqrt{Rs}$, using the most probable value of the coefficient of roughness.

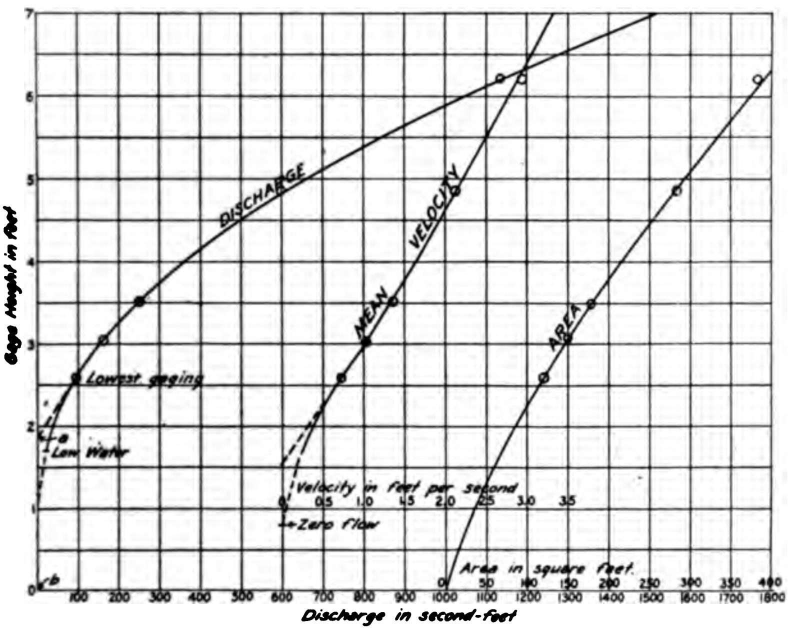


FIG. 17.—Typical Rating Curve, Showing Low-Water Extension.

The curve should be extended into low water with the greatest care. A slight variation from the true direction of the curve means a large percentage of error in the resulting estimate of minimum discharge. All conditions at the station should be studied. The curve must always be drawn to intersect the Y axis at the gage height of zero flow. If the point of zero flow is not known its true position will lie between the gage height of the bottom of the channel and the point where the tangent to the discharge curve at its lowest known value cuts the Y axis, as between a and b, fig. 17. If the mean velocity curve intersects the axis above the gage height of the bed of the stream—that is to say, if

there is ponded water—the curve will be convex to the Y axis; if it cuts the axis at the gage height of the bed of the stream the curve will be concave to the Y axis (fig. 17).

When measurements are not made at the gage—for example, when low-water measurements are made by wading—the discharge should be divided by the area of the section at the gage and the resulting velocity plotted on the velocity-curve. Points so found are useful in extending the velocity-curve into low water.

When the current is diagonal to the measured section the observed velocities should be reduced to velocities at right angles to the measured section, but the area should not be reduced. The area is a measured quantity, while the angle of the current is usually estimated and often varies with the stage.

STATION RATING CURVE.

Station rating curves which show graphically the discharge corresponding to any stage of the stream may be plotted either on ordinary or logarithmic cross-section paper. When ordinary cross-section paper is used the measurements of discharge are plotted either with discharge and gage heights as coordinates or with discharge and $A\sqrt{d}$ as coordinates, in which A is the area and d is the mean depth of the cross-section. When logarithmic cross-section paper is used, discharges and gage heights are the coordinates.

Ordinary cross-section paper with discharge and gage height as coordinates.—The usual method of constructing a rating curve for a gaging station is to plot the results of the discharge measurements on ordinary cross-section paper with gage heights and corresponding discharges as coordinates (fig. 18). The points so located define the position of a curve which is drawn among them. The horizontal and vertical scales will be so chosen that the curve may be used within the limits of accuracy for the work, and in its critical portions will make, as nearly as may be, angles of 45° with each axis. The discharge curve under natural conditions of free flow is always convex to the gage axis. Under special conditions, due to backwater, it may reverse at high stages and become concave to the gage axis.

If a sufficient number of accurate discharge measurements are available and are distributed throughout the range of stage, the discharge curve may be readily and accurately constructed. When such measurements are not available curves of reasonable accuracy may frequently be constructed by use of area and mean velocity curves or by one of the other methods of plotting.

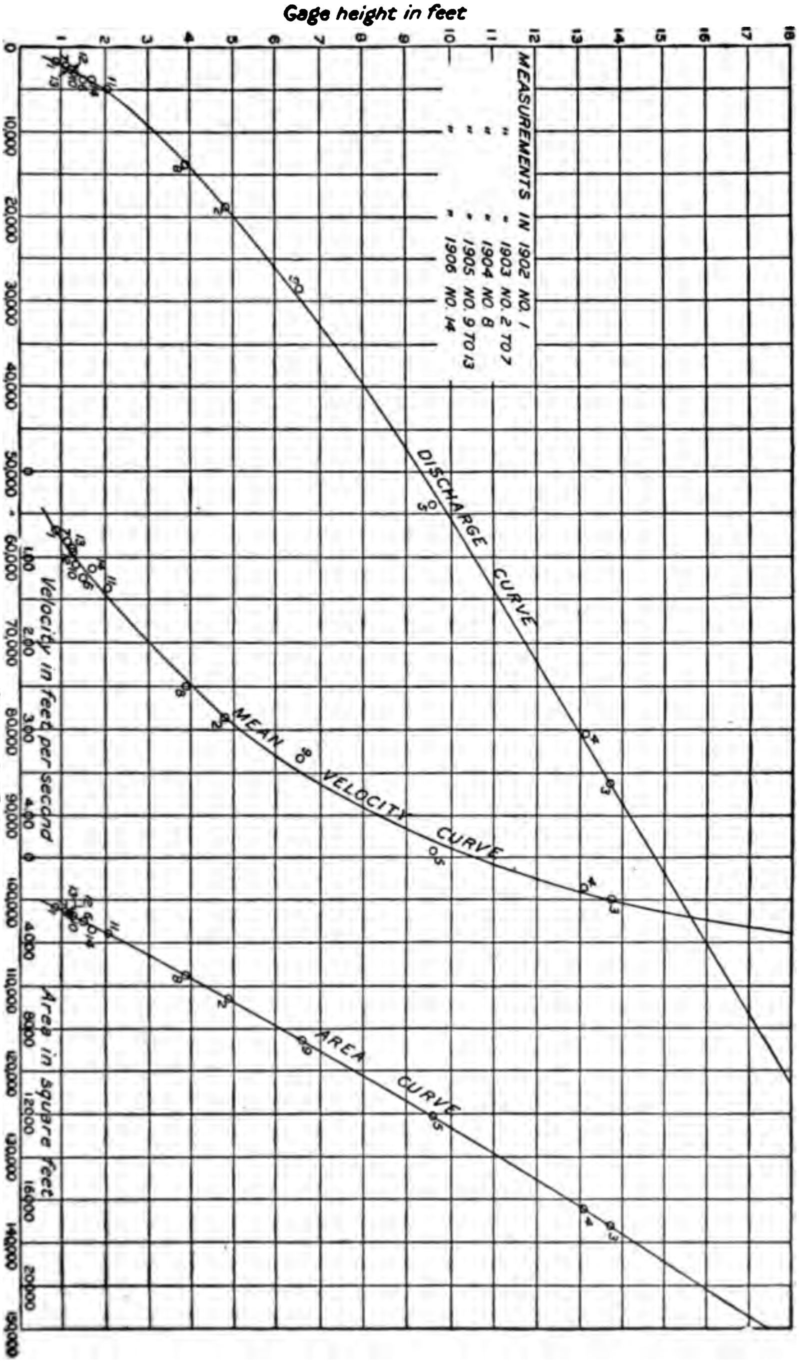


FIG. 18.—Discharge, Mean Velocity, and Area Curves, Potomac River at Point of Rocks Md.

In order to determine the accuracy of the individual measurements used in locating the station rating curve it is necessary to plot, as a function of the gage height, not only the discharge but also the mean velocity and area for each measurement. In this plotting the same gage-height scale should be used. The true area curve and approximate curves of discharge and mean velocity are then drawn through the points. The relation of the plotted points of discharge to the rating curve will show any discordant measurements. Whether the discord is due to erroneous area or velocity determinations will be shown by the relation of these respective points to the area and velocity curves, and the cause of any discrepancies in either factor can then be investigated. Such discrepancies may arise from error of observation or of computation.

The relative accuracy of the various plotted discharges having been determined, the rating curve can then be drawn, due weight being given to the various measurements. Points for portions of the curve not defined by actual discharge measurements can be determined by multiplying the area by the mean velocity as measured from the curves of area and velocity. For extending the rating curve either above or below the limits of the measurements the mean velocity and area curves may be constructed, as previously described, to the stages for which the discharge curve is desired, and the discharge curve found by taking the product of the two curves.

Whatever the method adopted in drawing the rating curve it should always be checked by computing the curve of mean velocity from the curves of area and discharge. If the curve of mean velocity so determined is not consistent with conditions at and near the station the discharge curve should be revised.

The discharge at a given stage of a rapidly rising stream is larger than for a falling or stationary stream at the same stage, as the surface slope, and hence the velocity, is greater for the first condition. This effect is but little noticed except during periods of extreme high water. At such times the water passes down the stream in a flood wave, and after the crest is passed a retarding effect may be caused which may reduce the slope practically to zero.

The curves shown in fig. 19 illustrate this. They are based upon the table of measurements on page 90. Therefore, in studying the plotted measurements, the fact whether the stream is rising, falling, or stationary should be considered. Inasmuch as rising stages are of much shorter duration than falling or stationary stages, more weight should

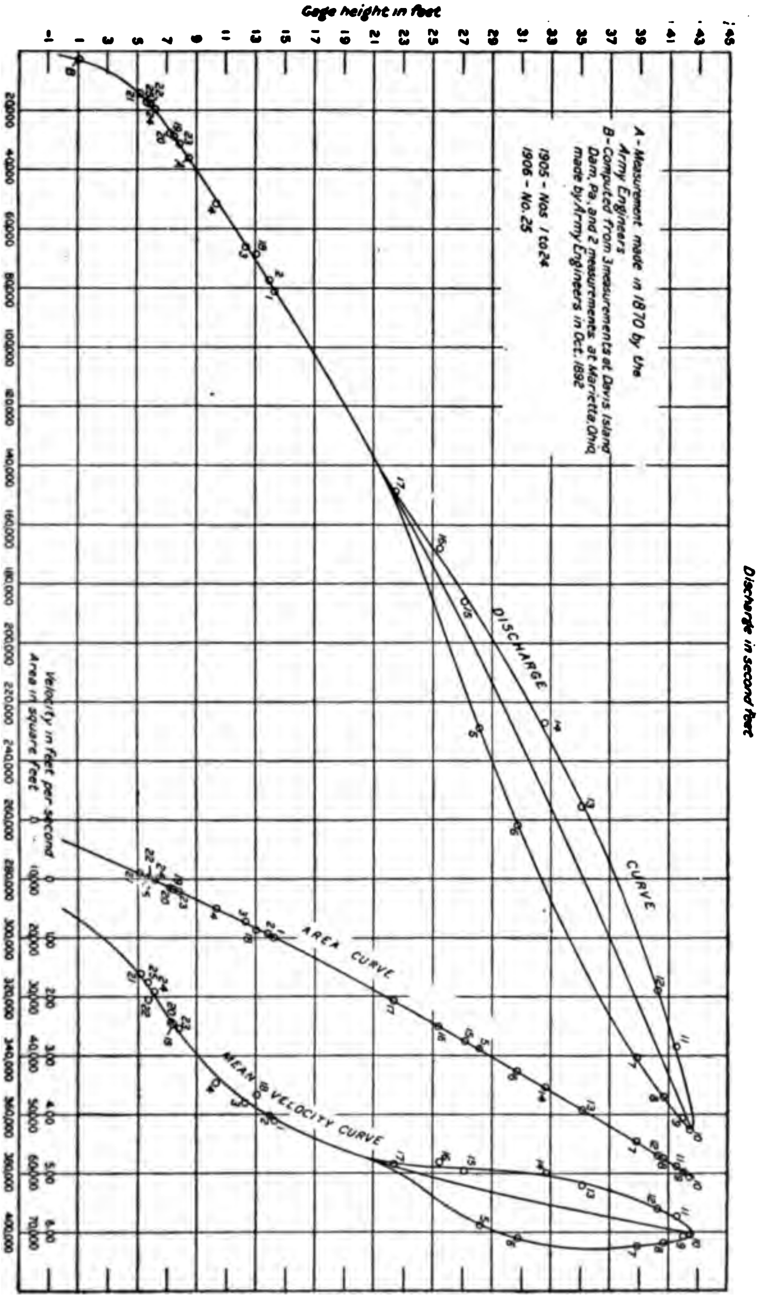


FIG. 19.—Discharge, Mean Velocity, and Area Curves, Ohio River at Wheeling, W. Va.

be given to measurements made on falling or stationary than on rising stages.

Discharge measurements of Ohio River at Wheeling, W. Va. Made in 1905 by E. C. Murphy.

No.	Date.	Area of section.	Mean velocity.	Gage height.	Change of stage.*	Discharge.
		Sq. ft.	*Ft. per sec.*	*Feet.*	*Feet.*	*Sec.-ft.*
5	March 20	38,890	5.89	28.2	+.68	229,200
6	" 20	42,750	6.13	30.8	+.60	261,900
7	" 21	54,780	6.23	38.9	+.37	341,100
8	" 21	57,360	6.18	40.7	+.20	354,400
9	" 22	59,580	6.07	42.05	+.05	361,600
10	" 22	60,510	6.05	42.5	+.05	365,700
11	" 23	58,830	5.73	41.6	−.20	336,900
12	" 23	56,790	5.60	40.3	−.27	318,100
13	" 24	49,250	5.20	35.2	−.35	255,800
14	" 24	45,550	4.99	32.7	−.40	227,300
15	" 25	37,560	4.95	27.2	−.23	186,100
16	" 25	35,050	4.80	25.5	−.14	168,100
17	" 27	30,830	4.83	22.44	−.05	149,100

* Rate of rise or fall per hour; rising +; falling —.

As the mean velocity and area curves, which are factorial curves in making the station rating curve, do not under ordinary conditions follow any mathematical law, the discharge curve will not generally be a mathematical curve. For ordinary streams it is made up of a series of parabolas. For many streams it approaches very nearly the form of a single parabola. Some engineers construct the rating curve by mathematical treatment, by use of least squares. In ordinary practice, however, this is not considered practicable, as the graphic method can be used with greater ease and speed and gives results as close as the data will justify.

If the engineer is familiar with the conditions in the channel at and near the station, a few careful measurements, well distributed, may serve to define the curve of mean velocity. If slope observations are taken and the point of zero flow is determined, a very good approximate rating can be made from two or three measurements.

Ordinary cross-section paper, with discharge and $A\sqrt{d}$ as coordinates.— In the construction of a rating curve based on a limited number of measurements, it is evident that it is much safer to extend a straight line than a curve. Investigations have consequently been made of the component parts of the discharge curve for a quantity which is readily measurable, and to which the discharge is approximately proportional, for use in conjunction with the discharge as a coordinate for plotting the discharge curve. The area times the square root of the mean depth of the stream, $A\sqrt{d}$, has been found by J. C. Stevens to be such a quantity.

From Kutter's formula $Q = Ac\sqrt{Rs}$ may be written $Q = (A\sqrt{R})(c\sqrt{s})$. If $(c\sqrt{s})$ is constant or approximately so, then Q varies directly as $(A\sqrt{R})$, and consequently when these two quantities are plotted as coordinates the result is a straight line. c is a function of s, R, and n. R increases with the stage. It is also a matter of observation that s in general increases with the stage, the relative change being small for high stages. For comparatively large slopes the effect of s on c is insignificant, or, to quote Trautwine, "for slopes greater than .01 the coefficient c is the same as for that slope." For flat slopes s has an appreciable effect on c. For a value of R greater than 3.28 feet or 1 meter, c

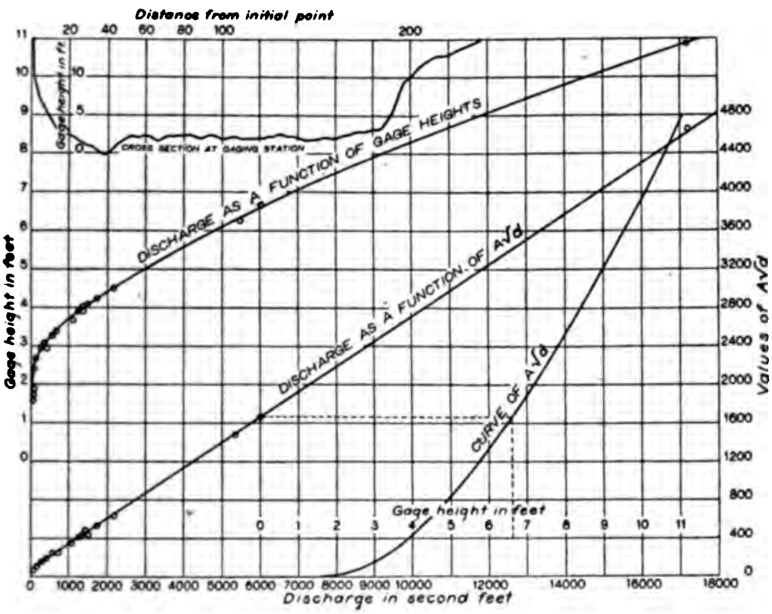


FIG. 20.—Rating Curve showing Discharge as a Function of $A\sqrt{d}$.

and s vary inversely, while c is of itself a decreasing function of s and an increasing function of R. Hence the product of $c\sqrt{s}$ may remain practically constant for a given set of conditions, but for values of R less than 3.28 feet, c is an increasing function of both s and R, and hence the product of $c\sqrt{s}$ is not a constant. The value of this method lies chiefly in making estimates for the higher stages and is not so generally applicable to shallow streams.

Based on the above conditions and assumptions, discharge curves may be plotted with Q and $A\sqrt{R}$ as coordinates. It has been found, however, that d, the mean depth of cross-section, can be substituted

for R and give practically the same results in plotting. It is also easier to determine d than R.

In the application of this method (fig. 20) plot the elevations of the bed of the stream above gage datum and thereby obtain a cross-section. From the cross-section prepare a table giving widths, areas, mean depths and values of $A\sqrt{d}$ for each foot or half-foot of gage heights. Widths may be scaled directly from the cross-section. The table of areas is quickly prepared by first computing the area for one gage height about midway of the range of stage. For increasing gage heights add successively the areas of trapezoids formed by the widths and gage-heights interval. For decreasing gage heights subtract these successive areas.

After the table of areas has been prepared the quantities $A\sqrt{d}$ $\left(\text{or } A\sqrt{\frac{A}{w}}\right)$, where w = width, can be read directly with a slide rule. On cross-section paper draw the curve of $A\sqrt{d}$, using gage heights as abscissas, as shown in the diagram. After this curve is drawn the values of $A\sqrt{d}$ are no longer required. Lay off a scale of discharge as abscissas. To plot a discharge measurement project from the horizontal scale of gage heights to the curve of $A\sqrt{d}$, thence horizontally to intersect the given discharge as indicated by dotted line. Points so plotted will generally conform to a straight line.

The illustration (fig. 20) also shows the station rating curve, in in which the same scale of discharges is used with gage heights as ordinates, shown on the left.

The straight line marked "discharge as a function of $A\sqrt{d}$" does not pass through the origin for reasons elsewhere stated as to the effect on the coefficient c of the rapidly changing slope at this stage. Therefore, when but a single measurement is at hand the line should be drawn to intersect the scale of $A\sqrt{d}$ at some point above the origin. This point has been found to correspond approximately to the gage height at which the mean depth of flowing water is between 1 and 2 feet.

In the case, frequently encountered, where there is ponded water at the gage height of zero discharge, the corresponding value of $A\sqrt{d}$ should be subtracted from the tabular values of this quantity before plotting. The gage height for which the discharge is zero can be determined by a careful examination, with levels or soundings, of the bed below the gaging section. Even in this case the straight line discharge curve will pass above the origin and should be treated as above outlined for conditions where ponded or dead water does not exist.

Logarithmic cross-section paper.—Cross-section paper graduated logarithmically may also be used in plotting the rating curve. On this

paper discharges and gage heights are plotted as coordinates. The curve resulting from the points so plotted is practically a straight line and has a corresponding advantage for extension. Logarithmic paper may give reliable results in the hands of an experienced operator, but if it is not properly and intelligently used large errors may arise. It is best used for channels with uniform conditions of cross-section and flow.

RATING OR DISCHARGE TABLE.

After the station rating curve has been constructed the next step in the computation of daily discharge is to prepare the station rating table, which gives the discharge of the stream at any stage. This table (see p. 94) will be constructed either for tenths, half-tenths, or hundredths of gage height, according to the readings of the gage to which it is to be applied. The table is made by first taking the discharges for various gage heights directly from the station rating curve. These discharges are then so adjusted that the differences for successive stages shall be either constant or gradually increasing, but never decreasing unless the station is affected by backwater.

The station rating table so obtained is applied to the gage heights to obtain the daily discharges.

The following tables and figs. 18 and 20 illustrate the method of determining daily discharge of streams with permanent beds:

Discharge measurements of Potomac River at Point of Rocks, Md., in 1902–7.

Date.	Hydrographer.	Area of section.	Mean velocity.	Gage height.	Discharge.
		Sq. ft.	*Ft. per sec.*	*Feet.*	*Sec.-ft.*
1902					
June 22	Newell and Paul	2,897	1.01	1.25	2,921
Sept. 2	E. G. Paul	2,356	.73	.87	1,717
1903					
Mar. 12	E. C. Murphy	6,600	2.86	4.84	18,880
Apr. 17	Hoyt and Paul	17,250	5.01	13.70	86,420
Apr. 17	Hoyt and Stokes	16,500	4.88	13.10	80,520
Apr. 18	Hoyt and Stokes	12,180	4.44	9.60	54,080
Sept. 14	Paul and Sawyer	2,950	1.28	1.50	3,770
Nov. 9	W. C. Sawyer	2,590	.83	1.12	2,140
1904					
July 11	Hoyt and Grover	5,500	2.50	3.87	13,750
1905					
Mar. 13	Tillinghast and Comstock	8,600	3.33	6.56	28,640
June 20	Grover and Lyman	2,727	1.10	1.29	2,997
Oct. 30	G. F. Harley	3,532	1.38	2.05	4,889
Nov. 9	G. F. Harley	2,703	.94	1.20	2,531
Nov. 9	Harley and Stewart	2,703	.91	1.20	2,467
1906					
May 30	R. Follansbee	3,351	1.16	1.70	3,892
Dec. 7	R. H. Bolster	3,180	1.40	1.76	4,450
1907					
Mar. 15	R. H. Bolster	21,460	5.31	16.95	114,000

Rating table for Potomac River at Point of Rocks, Md., from April 1, 1902, to December 31, 1906.

Gage height.	Discharge.	Difference.	Gage height.	Discharge.	Difference.	Gage height.	Discharge.	Difference.
Feet.	*Sec.-ft.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Sec.-ft.*
0.50	900	....	2.40	6,520	390	4.60	17,430	1,160
.60	1,090	190	.50	6,920	400	.80	18,610	1,180
.70	1,295	205	.60	7,330	410	5.00	19,820	1,210
.80	1,515	220	.70	7,750	420	.20	21,060	1,240
.90	1,750	235	.80	8,180	430	.40	22,300	1,240
1.00	2,000	250	.90	8,620	440	.60	23,560	1,260
10	2,260	260	3.00	9,070	450	.80	24,840	1,280
.20	2,530	270	.10	9,530	460	6.00	26,140	1 300
.30	2,810	280	.20	10,000	470	.20	27,460	1,320
.40	3,100	290	.30	10,480	480	.40	28,780	1,320
.50	3,400	300	.40	10,970	490	.60	30,100	1,320
.60	3,700	300	.50	11,470	500	.80	31,460	1,360
.70	4,010	310	.60	11,980	510	7.00	32,820	1,360
.80	4,330	320	.70	12,490	510	.50	36,340	3,520
.90	4,670	340	.80	13,010	520	8.00	39,980	3,640
2.00	5,020	350	.90	13,530	520	.50	43,740	3,760
.10	5,380	360	4.00	14,070	540	9.00	47,600	3,860
.20	5,750	370	.20	15,150	1,080	.50	51,560	3,960
.30	6,130	380	.40	16,270	1,120	10.00	55,600	4,040

Note: The above table is applicable only for open-channel conditions. It is based on discharge measurements made during 1902 to 1907. It is well defined between gage heights 1.0 feet and 17.0 feet. Above gage height 10 feet the rating curve is a tangent, the difference being 830 per tenth.

Daily gage heights and discharges of Potomac River at Point of Rocks, Md., for July to December, 1904.

Day.	July.		August.		September.		October.		November.		December.	
	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
	Feet.	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*
1	1.4	3,100	1.4	3,100	0.9	1,750	0.6	1,090	0.6	1,090	0.8	1,515
2	1.3	2,810	1.3	2,810	.8	1,515	.6	1,090	.6	1,090	.8	1,515
3	1.3	2,810	1.2	2,530	.8	1,515	.6	1,090	.7	1,295	.8	1,515
4	1.3	2,810	1.2	2,530	.8	1,515	.6	1,090	.7	1,295	.8	1,515
5	1.2	2,530	1.2	2,530	.9	1,750	.6	1,090	.7	1,295	.8	1,515
6	1.5	3,400	1.2	2,530	1.0	2,000	.6	1,090	.7	1,295	.8	1,515
7	1.5	3,400	1.3	2,810	.9	1,750	.6	1,090	.7	1,295	.8	1,515
8	1.6	3,700	1.4	3,100	.8	1,515	.6	1,090	.7	1,295	.8	1,515
9	1.6	3,700	1.5	3,400	.7	1,295	.5	900	.7	1,295	.8	1,515
10	1.7	4,010	1.4	3,100	.7	1,295	.5	900	.7	1,295	.9	1,750
11	2.9	8,620	1.3	2,810	.7	1,295	.5	900	.7	1,295	.9	1,750
12	2.6	7,330	1.3	2,810	.8	1,515	.6	1,090	.7	1,295	.9	1,750
13	3.4	10,970	1.2	2,530	1.0	2,000	.7	1,295	.7	1,295	.9	1,750
14	3.1	9,530	1.2	2,530	.9	1,750	.7	1,295	.8	1,515	.9	1,750
15	3.0	9,070	1.1	2,260	.9	1,750	.6	1,090	.8	1,515	.9	1,750
16	2.8	8,180	1.1	2,260	1.0	2,000	.6	1,090	.8	1,515	.9	1,750
17	2.4	6,520	1.1	2,260	1.0	2,000	.5	900	.7	1,295	.9	1,750
18	2.0	5,020	1.0	2,000	.9	1,750	.5	900	.8	1,515	.9	1,750
19	1.8	4,330	1.0	2,000	.8	1,515	.5	900	.8	1,515	.9	1,750
20	1.6	3,700	1.0	2,000	.7	1,295	.6	1,090	.8	1,515	1.0	2,000
21	1.4	3,100	1.0	2,000	.8	1,515	1.0	2,000	.7	1,295	1.0	2,000
22	1.4	3,100	.9	1,750	1.0	2,000	.9	1,750	.7	1,295	1.0	2,000
23	1.3	2,810	.9	1,750	.9	1,750	.8	1,515	.7	1,295	1.0	2,000
24	1.3	2,810	1.0	2,000	.8	1,515	.7	1,295	.7	1,295	1.0	2,000
25	1.3	2,810	1.0	2,000	.8	1,515	.7	1,295	.7	1,295	1.1	2,260
26	1.4	3,100	1.2	2,530	.8	1,515	.7	1,295	.7	1,295	1.4	3,100
27	1.4	3,100	1.1	2,260	.7	1,295	.7	1,295	.7	1,295	1.5	3,400
28	1.4	3,100	1.1	2,260	.7	1,295	.7	1,295	.7	1,295	1.8	4,330
29	1.5	3,400	1.0	2,000	.7	1,295	.6	1,090	.8	1,515	1.8	4,330
30	1.5	3,400	1.0	2,000	.7	1,295	.6	1,090	.8	1,515	1.9	4,670
31	1.5	3,400	.9	1,750	....	....	.6	1,090	....	....	2.0	5,020
Total...		139,670		74,200		47,760		36,080		40,200		68,245
Mean...		4,505		2,394		1,592		1,164		1,340		2,201

Monthly discharge of Potomac River at Point of Rocks, Md., for 1904.

Month.	Discharge in second-feet.				Run-off.	
	Maximum.	Minimum.	Mean.	Second-feet per sq. mi.	Depth in inches.	Acre-feet.
July	10,970	2,530	4,505	.467	.538	277,300
August	3,400	1,750	2,394	.248	.538	147,200
September	2,000	1,295	1,592	.165	.184	94,730
October	2,000	900	1,164	.121	.140	71,570
November	1,515	1,090	1,340	.139	.155	79,740
December	5,020	1,515	2,201	.228	.263	135,300
The period	10,970	900	2,199	.228	1.556	805,840

STATIONS ON STREAMS WITH CHANGEABLE BEDS.

The determination of the daily discharge of streams with changeable beds is more difficult than of those with permanent beds. The method used varies with the rapidity of the changes. The base data for such determinations are the same as those used for permanent beds, but more frequent discharge measurements are necessary, as otherwise the results obtained are only roughly approximate.

PERIODICALLY CHANGING BEDS.

For stations with beds which shift slowly or are changed only during floods, station rating curves can be prepared as above described for periods between changes, and satisfactory results can be obtained with two or three measurements a month, provided measurements are taken soon after such changes take place.

CONSTANTLY CHANGING BEDS.

For streams with continually shifting beds, as the Colorado and Rio Grande, discharge measurements should be made every two or three days and the discharge for the intervening days estimated by interpolation, modified by the gage heights for these days. There are two methods of making these interpolations, the Stout and the Bolster methods, known by the names of their inventors.

Stout method.—In the Stout method an approximate station rating curve and rating table are prepared from the discharge measurements and applied to modified or so-called corrected daily gage heights. The gage heights are corrected by means of a curve (fig. 21) determined by plotting as ordinates the differences between the actual gage heights at the time of the various discharge measurements and the gage height

corresponding on the approximate curve to the respective measured discharges, and as abscissas the corresponding days of the months. Through these points an irregular curve is drawn, from which can be found the correction for days other than those on which measurements were made. The correction is positive if the discharge is greater than that given by the station rating curve, negative if less. Each daily gage height is then corrected by the amount indicated on the correction curve, and the discharge corresponding thereto is taken from the approximate rating table.

Bolster method.—In the Bolster method the discharge measurements for the entire year are first plotted with discharges as abscissas and gage heights as ordinates. The points so plotted are considered chronologically and, even though scattered, will usually locate one or more fairly well-defined curves, called standard curves (fig. 21). In general the number and position of these standard curves is determined by the radical changes in the stream bed due to floods.

When beds change very rapidly it is necessary to change the position of the rating curve from day to day, making practically a new curve daily. This daily curve is of the same form as the standard curve and is parallel to it with respect to ordinates. For a day when a measurement is made the rating curve passes through such plotted measurement. In order to locate a rating curve for other days a line connecting consecutive measurements is drawn and divided into as many equal parts as there are days intervening between the measurements, on the assumption that the change in conditions of flow between any two consecutive measurements is uniform from day to day. The daily rating curve will then pass through these points of division, and the discharge is read directly from these curves by applying to them the observed daily gage heights.

In order to facilitate the use of the method and to make it as rapid in application as the common method for permanent stations the standard curve or curves, together with a vertical line of reference, should be transferred from the original station sheet to tracing cloth, which can be readily shifted vertically to any desired position by always keeping the two reference lines coincident with each other.

In applying and modifying this method judgment must be used for long time intervals of no measurements, or for radical changes in the stream bed caused by sudden floods. The tables on pages 98–99 and fig. 21 illustrate this method of obtaining daily discharge.

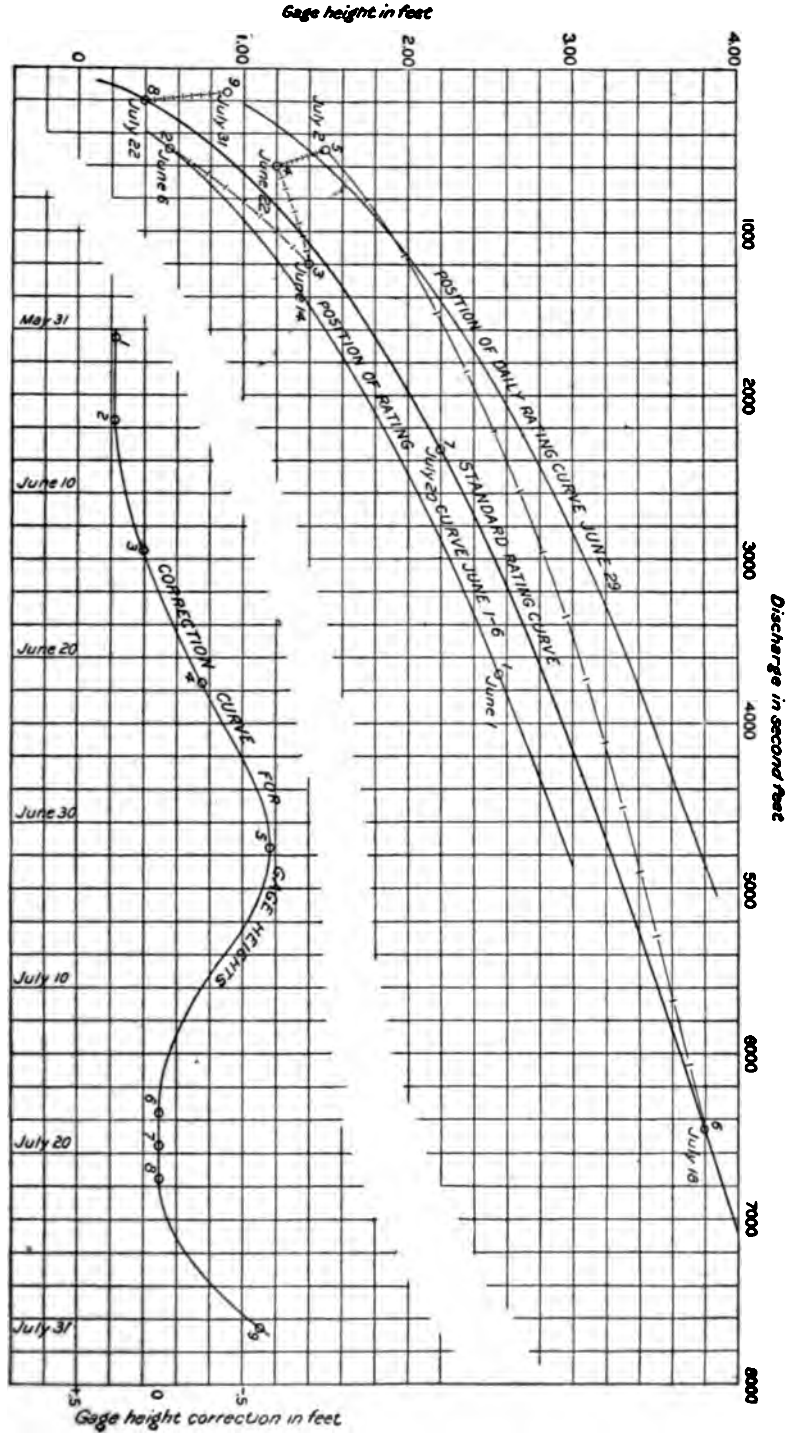


FIG. 21.—Typical Curves Illustrating the Stout and the Bolster Methods of Computing Daily Flow.

List of measurements to illustrate the Stout and the Bolster methods of determining daily discharge.

No.	Date.	Gage height.	Discharge.
		Feet.	*Sec.-ft.*
1	June 1	2.55	3700
2	6	55	500
3	14	1.4	1200
4	22	1.2	600
5	July 2	1.5	500
6	18	3.8	6460
7	20	2.2	2330
8	22	.4	200
9	31	.9	150

Daily gage heights and discharges to illustrate the Stout and the Bolster methods of determining daily discharge.

Day.	June.				July.			
	Observed gage heights.	Discharge, Bolster method.	Corrected gage heights.	Discharge, Stout method.	Observed gage heights.	Discharge, Bolster method.	Corrected gage heights.	Discharge, Stout method.
	Feet.	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*
1	3.0	4870	3.3	4970	1.5	530	.85	520
2	2.5	3580	2.8	3650	1.5	500	.85	520
3	2.0	2490	2.3	2530	1.5	570	.85	520
4	1.5	1610	1.8	1640	1.6	730	.95	605
5	1.0	930	1.3	980	1.6	780	1.0	650
6	.4	390	65	365	1.8	1050	1.25	905
7	.6	510	.85	520	1.7	970	1.2	850
8	.6	490	85	520	1.7	1020	1.25	905
9	7	550	.95	605	1.9	1330	1.55	1280
10	8	620	1.0	650	2.0	1510	1.7	1490
11	1.0	800	1.2	850	2.2	1890	2.0	1970
12	1.5	1400	1.65	1420	2.6	2720	2.45	2850
13	1.4	1240	1.55	1280	3.0	3700	2.9	3900
14	1.4	1200	1.5	1210	3.6	5440	3.5	5550
15	1.3	1020	1.35	1020	3.7	5850	3.65	6000
16	1.1	760	1.15	800	3.5	5370	3.5	5550
17	1.0	620	1.0	650	3.2	4610	3.2	4690
18	1.0	580	95	605	3.7	6150	3.7	6150
19	1.1	640	1.0	650	4.0	7080	4.0	7080
20	1.2	690	1.05	700	3.0	4160	3.0	4160
21	1.2	650	1.0	650	1.8	1640	1.8	1640
22	1.3	690	1.05	700	.8	480	.8	480
23	1.5	850	1.2	850	.3	120	.3	150
24	1.8	1170	1.45	1140	.3	100	.25	130
25	1.7	990	1.3	960	.4	110	.35	175
26	1.6	830	1.15	800	.4	90	.3	150
27	1.6	780	1.1	750	.6	130	.4	200
28	1.6	740	1.05	700	.7	150	.45	230
29	1.6	700	1.0	650	.8	170	.45	230
30	1.6	660	.95	605	.9	180	.4	200
31	....	....	....	....	1.0	200	.4	200
Totals.		33050		33400		59320		59930
Means.		1102		1113		1914		1933

Rating table to illustrate the Stout and the Bolster methods of determining daily discharge.

Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.	Gage height.	Discharge.
Feet.	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*	*Feet.*	*Sec.-ft.*
0.00	60	1.00	650	2.00	1970	3.00	4160	4.00	7080
.10	80	.10	750	.10	2150	.10	4420	......	......
.20	110	.20	850	.20	2330	.20	4690	......	......
.30	150	.30	960	.30	2530	.30	4970	......	......
.40	200	.40	1080	.40	2740	.40	5260	......	......
.50	260	.50	1210	.50	2960	.50	5550	......	......
.60	330	.60	1350	.60	3180	.60	5850	......	......
.70	400	.70	1490	.70	3410	.70	6150	......	......
.80	480	.80	1640	.80	3650	.80	6460	......	......
.90	560	.90	1800	.90	3900	.90	6770	......	......

ICE-COVERED STREAMS.

The daily discharge of an ice-covered stream is computed by the general method outlined above for open streams. It is necessary, however, to construct a special station rating curve (fig. 22) from measurements of discharge made during the ice period. This is accomplished by using as coordinates the measured discharges and corresponding gage heights. Two curves should be constructed by using two gage heights for each discharge measurement, the gage height to the surface of the water as it rises in a hole cut through the ice and the gage height of the bottom of the ice. That curve which is best defined should then be used for the computation of daily discharges or other values.

These and other curves indicate that somewhat better results are obtained by using gage heights to the water surface. There is no great difference, however, except for low stages.

The computation of the daily discharge by the application of the station rating table for open water to the gage heights during the ice period and the use of a coefficient will not give accurate or satisfactory results. The coefficient necessary for such use may range from .35 or less for very heavy ice on relatively shallow streams to .80 or more for thin ice on deep streams.

All records for the frozen season should be scrutinized with great care in order to determine, if possible, whether there have been disturbing influences, such as ice jams, below the gage, which have so affected the record of stage that the rating table is not applicable to it. Even after this care has been taken, the computed discharges should be compared with other records of flow in the same section as a check upon them (see pp. 65–69).

COMPUTATION OF OTHER VALUES OF DISCHARGE AND RUN-OFF.

The quantity of water flowing in a stream is expressed by various terms or units, each of which is associated with a certain class of work. These terms may be divided into two classes—those which represent a rate of flow, or the discharge, as second-feet, gallons per minute, miner's inch, and second-feet per square mile; and those which represent actual quantities of water, or the run-off, as depth in inches and acre-feet.

UNITS OF DISCHARGE.

The second-foot.—The usual term for expressing discharge is the second-foot. As the second-foot is a unit of rate of flow and is in itself indefinite as regards duration, it must generally be used in connection

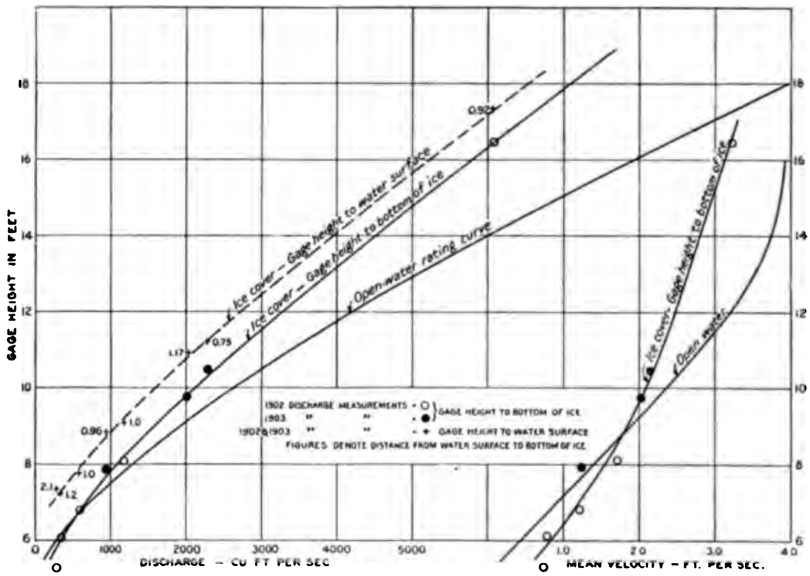


FIG. 22.—Rating and Velocity Curves under Ice, Wallkill Creek at Newpaltz, N. Y.

with some unit of time, as a day, week, month, or year, in which event the term means that for each second during the period of time selected the flow averaged so many cubic feet per second. This has generally been considerered the fundamental unit of discharge and from it other terms of discharge and run-off are usually computed, the conversion to other units being a simple arithmetical process, usually made by means of tables specially prepared for the purpose.

Gallons per minute.—Gallons per minute is generally used in connection with pumping and city water supply.

Miner's inch.—The miner's inch is usually defined as the quantity of water which passes through an orifice one inch square under a fixed head, which varies locally. It has been commonly used by miners and irrigators throughout the West, and is defined by statute in many States. Owing to the confusion caused in measuring the miner's inch, and to the fact that as formerly defined by size of orifice and head it was not exact, it is now defined as a certain part of a second-foot, usually $\frac{1}{50}$ or $\frac{1}{40}$.

Second-feet per square mile.—Second-feet per square mile is the average number of cubic feet of water flowing per second from each square mile of area drained, on the assumption that the discharge is distributed uniformly both as regards time and area. It is found by dividing the mean discharge in second-feet by the drainage area.

UNITS OF RUN-OFF.

Run-off in inches.—Run-off in inches is the depth to which a plane surface equal in extent to the drainage area would be covered if all the water flowing from it in a given period were conserved and uniformly distributed thereon. It is used for comparing run-off with rainfall, which is usually expressed in depth in inches.

Acre-foot.—An acre-foot is equivalent to 43,560 cubic feet, and is the quantity required to cover an acre to the depth of one foot. It is commonly used in connection with storage. There is a convenient relation between the second-foot and the acre-foot. One second-foot flowing for twenty-four hours will deliver 86,400 cubic feet, or approximately 2 acre-feet. One acre-foot equals 325,851 gallons, or a million gallons is somewhat more than 3 acre-feet.

On pages 127–129 are given conversion tables for the various units of discharge and run-off.

ACCURACY OF STREAM-GAGING DATA.[a]

In considering the accuracy that can be expected and obtained in stream-gaging data, it must be remembered that both the total flow of a stream and its distribution over the drainage area are constantly changing and that exact conditions existing at any time may probably never occur again. Therefore the flow that may be expected in the future for a given stream can be determined only by a study of a series of records extending over a long time. From these records it is possible to obtain

[a] See U. S. Geol. Survey Water-Supply Paper No. 95.

information which will show the conditions of flow to be expected in the future. Records of a reasonable degree of accuracy that extend over a considerable period of time are consequently of much more value than extremely accurate data for a short period.

The sources of error in estimates of stream flow may be divided into four classes. Most of these errors are readily within the control of the engineer and can be eliminated by careful work. A few, however, can be eliminated only by the expenditure of considerable time and money. No effort is made to discuss in detail the effect of these various errors except those pertaining to the current meter and its use. A list of the errors is given in order that the engineer may know where to investigate in case of discordant results.

1. Gage height errors, due to—
   (*a*) Change of relation between the gage reading and true height above gage datum.
   (*b*) Error in reading gage by observer.
   (*c*) Fluctuation of water stage between readings.
   (*d*) Error due to reading to tenths.
2. Errors of measurement:
   (*a*) Instrumental errors.
   (*b*) Velocity errors.
   (*c*) Area errors.
   (*d*) Computation errors.
3. Errors due to changes in conditions:
   (*a*) Ice conditions.
   (*b*) Shifting stream bed.
   (*c*) Backwater effect of dams.
   (*d*) Temporary obstructions.
4. Computation errors:
   (*a*) Errors due to fundamental assumptions.
   (*b*) Errors of construction and extension of curves.
   (*c*) Errors due to incorrect period of application of curve.
   (*d*) Errors due to changed condition of flow (which may or may not be shown by measurement).

A large number of comparisons have been made by engineers to determine the accuracy obtainable by the current meter. These comparisons have been made between the meter and either a calibrated tank or a sharp-crested weir, which are the two best methods known for measuring discharge in open channels.

Among these experiments three series of comparisons between the current meter and sharp-crested weirs are of special interest, namely,

those by A. Fteley in the Sudbury conduit; those by E. C. Murphy in the Cornell hydraulic laboratory canal; and those by H. K. Barrows on Westfield Little River, Massachusetts. The first two series are a direct comparison of current meter and weir measurements, while the third is a comparison of the daily discharge obtained during a period of time by means of current-meter measurements, daily gage heights, and a station rating curve, and those obtained by the weir. The results of these various experiments are given below. These and many other experiments show that when properly manipulated the current meter can be fully relied upon for gaging streams.

Comparisons by Mr. Fteley were made in the Sudbury conduit at the outlet of Farm Pond, Massachusetts, with the Fteley current meter, and are fully described in the Transactions of the American Society of Civil Engineers, Volume XII, page 301. A summary of the results is shown in the following table:

No. of experiment.	Percentage of difference of current-meter from weir measurement.
20	−9.4
26	−6.5
27	−5.6
17	−6.5
19	−4.5
24	−3.4
25	−2.3
15	−3.6
16	−1.8
23	−2.3

Mr. Murphy's comparisons were made in the canal of the Cornell hydraulic laboratory with the Haskell and Price meters. The discharges ranged from 197 to 230 second-feet. This work is fully described in U. S. Geological Survey Water-Supply Paper No. 95. A summary of the results is given in the following table:

No. of experiment.	Kind of meter.	Percentage of difference of current meter from weir measurements.			
		Max.	Min.	Mean.	Range.
		Per cent.	*Per cent.*	*Per cent.*	*Per cent.*
20	Haskell, No. 3	−4.73	0.00	−0.33	−4.73 to +2.86
20	S. Price, No. 363	−3.87	0.01	−0.76	−3.87 to +1.27
18	S. Price, No. 351	−1.89	−0.13	−1.38	−1.89 to −0.13
18	S. Price, No. 363	−2.77	+0.23	−1.56	−2.77 to +0.23
22	S. Price, No. 351	−4.60	−2.07	−2.78	−4.60 to −2.07
22	Haskell, No. 3	+0.85	−0.27	+0.40	+0.85 to −0.27

During the latter part of 1906 a current-meter and a weir station were maintained in conjunction on Westfield Little River, Massachusetts

and the mean daily flow as shown in the following table, with the percentages of difference, has been determined by the two methods. At the current-meter station a daily rating table was prepared from the discharge measurements and the daily discharges were obtained by applying this rating table to the daily gage heights. At the weir station the daily discharges were obtained by the use of the weir formula.

Daily discharge, in second-feet, of Westfield Little River near Blandford, Mass., 1906, as determined by weir and by current meter ratings.

Day.	Aug.			Sept.			Oct.			Nov.		
	Weir.	Current meter.	Per cent difference.	Weir.	Current meter.	Per cent difference.	Weir.	Current meter.	Per cent difference.	Weir.	Current meter.	Per cent difference.
1				5.42	5.8	+ 7	18.9	18.7	− 1	21.5	23.3	+ 8
2				4.86	4.8	− 1	16.5	15.2	− 8	20.5	21.1	+ 3
3				7.81	8.0	+ 2	13.3	11.9	−12	19.4	20.1	+ 3
4				7.81	8.7	+11	11.9	10.3	−16	17.2	16.4	− 5
5				6.58	7.3	+11	9.11	8.2	−11	16.3	15.6	− 4
6				5.42	5.8	+ 7	7.81	7.3	− 7	16.3	15.2	− 7
7				4.86	4.8	− 1	10.5	9.0	−17	15.8	14.4	−10
8				4.33	4.4	+ 2	9.11	8.0	−14	16.1	12.9	−25
9				3.82	4.1	+ 7	10.5	9.8	− 7	15.0	12.9	−16
10	21.5	23.9	+11	3.32	3.9	+17	56.6	67.3	+ 9	13.3	11.6	−15
11	19.8	22.8	+15	2.85	3.6	+26	33.0	40.9	+24	22.6	19.6	−15
12	18.1	17.8	− 2	4.33	4.6	+ 6	25.2	30.7	+22	36.1	33.8	− 7
13	12.6	15.2	+20	5.42	5.2	− 4	19.8	19.6	− 1	38.5	35.5	− 8
14	11.9	11.6	− 3	3.82	4.4	+15	17.3	12.9	−34	31.6	31.5	0
15	10.5	10.6	+ 1	3.32	3.7	+11	11.9	10.9	− 9	28.3	25.8	−10
16	9.11	9.8	+ 7	2.85	3.1	+ 8	11.9	10.3	−15	29.0	24.5	−18
17	7.81	8.0	+ 2	2.85	3.1	+ 8	10.5	9.2	−14	26.1	22.8	−14
18	7.81	8.2	+ 5	2.40	2.8	+17	11.9	11.6	− 3	56.6	53.1	− 6
19	6.58	6.9	+ 5	2.40	2.8	+17	11.9	11.2	− 6	a367	367	0
20	5.42	6.0	+11	4.33	4.6	+ 6	a297	297	0	193	234	+21
21	26.1	30.7	+17	25.2	22.8	−10	150	167	+11	128	139	+ 8
22	17.3	16.0	− 8	31.0	27.8	−11	84.0	86.5	+ 3	122	139	+14
23	11.2	10.9	− 3	35.0	38.1	+ 9	60.3	56.5	− 7	86.8	93.9	+ 8
24	13.3	14.0	+ 5	14.9	13.6	−10	44.8	42.8	− 5	65.4	72.5	+10
25	10.5	9.8	− 7	11.2	10.3	− 9	86.8	82.2	− 5	50.7	49.9	− 2
26	7.81	8.7	+11	9.11	8.7	− 5	77.4	80.7	+ 4	46.0	46.8	+ 2
27	7.20	8.0	+11	13.3	10.3	−29	53.1	54.2	+ 2	44.8	46.8	+ 4
28	18.9	18.7	− 1	9.11	9.0	− 1	41.5	44.8	+ 8	46.0	46.8	+ 2
29	11.9	11.9	0	10.5	9.8	− 7	35.0	37.3	+ 6	39.3	39.0	− 1
30	9.11	9.0	− 1	25.2	25.1	0	31.0	29.2	− 6	41.5	42.8	+ 3
31	6.58	7.8	+18				29.0	27.8	− 4			
Mean	12.3	13.0	+ 5	9.1	9.0	− 1	42.2	42.9	+ 2	55.7	57.6	+ 3

a The maxima for October and November are by the current meter. The weir was not large enough to measure these discharges.

DETERMINATION OF DURATION OF FLOW AND HORSEPOWER.

One of the first steps in investigations for developments which will utilize the flow of streams is the determination of the water supply that can be expected. This includes data in regard to the absolute minimum flow, the average minimum flow, and the maximum flow. For each

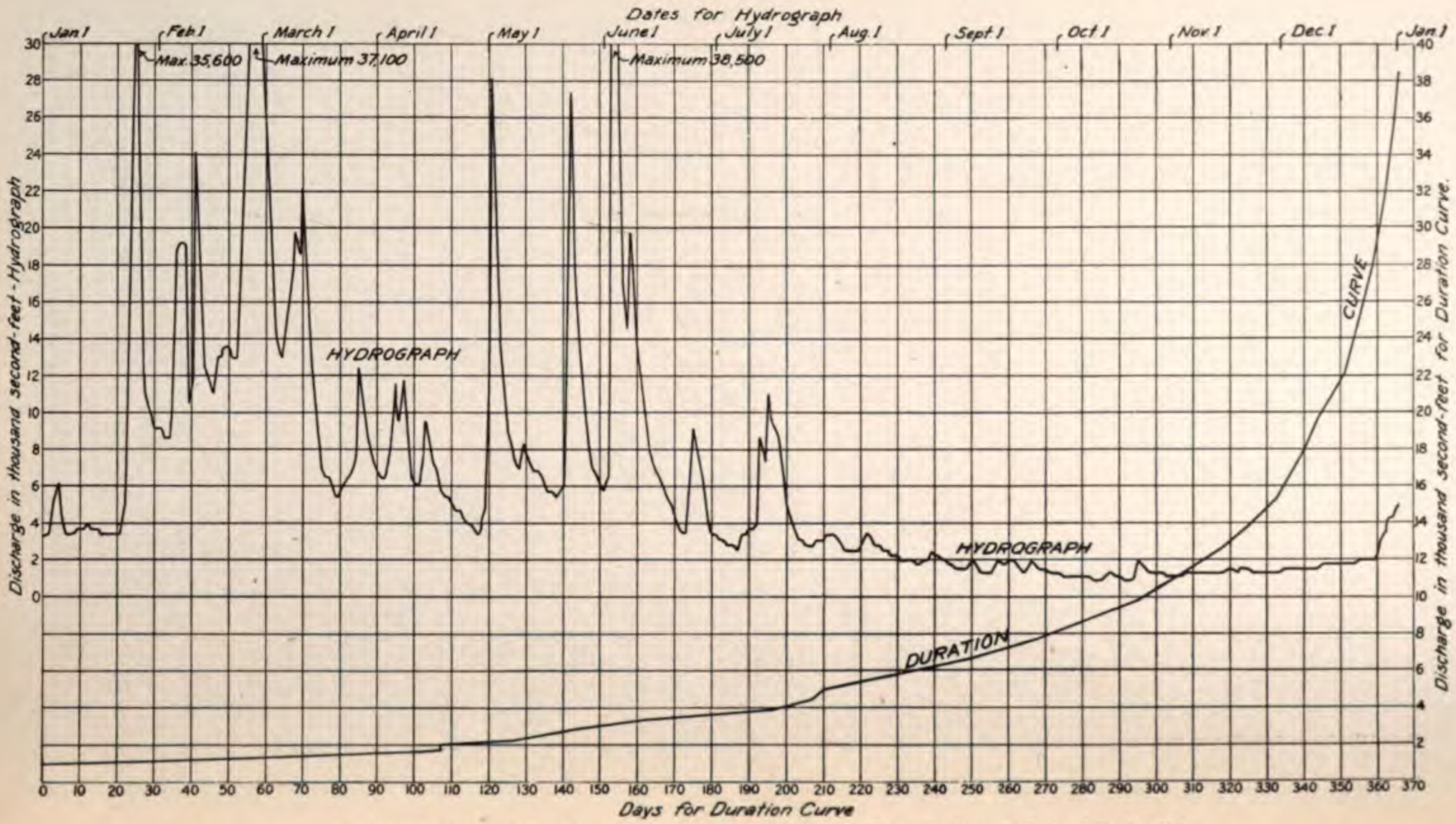


FIG. 23.—Hydrograph and Curve of Duration of Flow for 1904, Potomac River at Point of Rocks, Md.

case the duration as well as the quantity of the flow is essential. In order to obtain a reliable idea of the conditions of flow that may be expected, several years' records of daily flow should be available for study and comparison.

The duration of flow for a given year may be determined by tabulating, as shown in the following table, the values of the various daily flows in order of their magnitude, and then tabulating the number of days of the year on which each flow occurs. The sum of the numbers in the "Number of days" column up to any given flow will give the number of days when the flow is less than that indicated in the "Discharge" column, or the number of days deficiency. The column should be filled out showing deficiency for each discharge.

By plotting the discharge as abscissas and the number of days deficiencies as ordinates, a curve (fig. 23) can be constructed which shows the number of days during the year when the discharge is below any given amount.

The horsepower corresponding to the various discharges can then be computed and tabulated on the sheets with the discharge, which will show the number of days when an auxiliary will have to be used for given development. In computing the horsepower the loss of head caused by backwater, if any, should be taken into account.

Discharge and horsepower table for Potomac River at Point of Rocks, Md., for 1904.

Discharge in sec.-ft.	Horsepower (80% eff.) per foot fall.	Number days duration between consecutive values of discharge in first column.													Days of deficient discharge.
		Jan.	Feb.	March.	April.	May.	June.	July.	August.	Sept.	Oct.	Nov.	Dec.	Totals.	
900	82	..	..	..	..	..	..	..	..	..	0	..	..	0	....
990	90	..	..	..	..	..	..	..	..	..	6	..	..	6	6
1,100	100	..	..	..	..	..	..	..	..	..	15	2	..	17	23
1,320	120	..	..	..	..	..	..	..	..	8	7	20	..	35	58
1,540	140	..	..	..	..	..	..	..	..	10	1	8	9	28	86
1,760	160	..	..	..	..	..	..	..	3	7	1	..	10	21	107
1,980	180	..	..	..	..	..	..	..	..	..	..	..	..	...	....
2,200	200	..	..	..	..	..	..	..	8	5	1	..	5	19	126
2,750	250	..	..	..	..	..	..	1	12	..	..	..	1	14	140
3,300	300	..	..	..	..	..	1	12	7	..	..	..	1	21	161
3,850	350	17	..	..	3	..	5	8	1	..	..	..	1	35	196
4,400	400	2	..	..	3	..	2	2	..	..	..	..	2	11	207
4,950	450	..	..	..	2	..	..	..	..	..	..	..	1	3	210
5,500	500	2	..	1	4	1	3	1	..	..	..	..	1	13	223
6,600	600	1	..	7	5	10	2	1	..	..	..	..	..	26	249
7,700	700	..	..	4	4	6	4	1	..	..	..	..	..	19	268
8,800	800	..	2	3	1	4	2	2	..	..	..	..	..	14	282
9,900	900	2	1	1	4	3	1	2	..	..	..	..	..	14	296
11,000	1,000	2	3	2	..	..	1	1	..	..	..	..	..	9	305
13,200	1,200	1	7	3	3	2	1	..	..	..	..	..	..	17	322
15,400	1,400	1	3	3	..	2	2	..	..	..	..	..	..	11	333
17,600	1,600	..	1	3	..	..	2	..	..	..	..	..	..	6	339
19,800	1,800	..	3	1	..	1	..	..	..	..	..	..	..	5	344
22,000	2,000	2	1	2	..	..	2	..	..	..	..	..	..	7	351
27,500	2,500	..	4	1	..	2	..	..	..	..	..	..	..	7	358
33,000	3,000	..	3	..	1	..	1	..	..	..	..	..	..	5	363
38,500	3,500	1	1	..	..	..	1	..	..	..	..	..	..	3	366
Total days........		31	29	31	30	31	30	31	31	30	31	30	31	366	....

SUGGESTIONS FOR ESTIMATING DISCHARGE.

The engineer is often called upon for estimates of flow of streams on which few if any measurements of discharge have been made. In such cases it is necessary to prepare estimates of discharge and run-off for the stream basin by comparison of that basin with others for which records are available.

When no measurements of discharge are available the estimates are made by determining from records at other stations the probable discharge and run-off per square mile from the area under consideration. This multiplied by the drainage area gives the discharge. Such comparisons can be safely made, however, only when the streams used are in the same section of the country and are similar in character. The table, pp. 10-13, gives the run-off per square mile from areas in the northeastern United States.

When few measurements are available coefficients may be determined by means of which the discharge can be computed from the records at a neighboring station.

Rainfall data are of use as a check on estimates of flow, and they also show years of high and low water. As noted in Chapter II, care should be exercised in their use.

HYDROGRAPHS.

In order to show graphically the daily and seasonal distribution of flow of a stream for comparative purposes, hydrographs (fig. 23) are prepared by plotting the daily discharge for each day during the year and connecting the points so plotted by a curve. These curves are of use not only in studying the variations in flow from year to year, but may also be used in storage problems, where the total quantity of water is an essential factor. The mass curve may also be used to advantage in the study of storage. (See Water-Supply Paper No. 197, U. S. Geol. Survey.)

WHERE STREAM-GAGING DATA CAN BE FOUND.[a]

As a result of hydrographic studies by the Federal Government, by States, by special commissions, and by individuals, data in regard to stream flow and the conditions affecting it are now available for streams in nearly all sections of the country. These data are contained in publications which should form a part of every engineer's library, and should be freely used in order to avoid duplication of work.

[a] Notes on Hydrology, by Daniel W. Mead, professor of hydraulic engineering, Wisconsin University, contains many tables and references in regard to hydrologic data.

These publications are prepared and issued by—

1. United States Geological Survey.
2. United States Census.
3. United States Weather Bureau.
4. Corps of Engineers, United States Army.
5. State officials.
6. Special commissions.
7. City officials.

Reports of the United States Geological Survey.—The United States Geological Survey has for many years carried on systematic measurements of flow of streams and publishes annually a report of the results of such stream measurements. In connection with this work much other data, such as river profiles, quality of water, etc., are collected, and from time to time as information becomes available, special reports are prepared, which either bring together all the hydrographic data for particular drainage areas, or treat of special hydrographic subjects. These are published in the series of Water-Supply and Irrigation Papers.

Reports of the United States Census.—Volumes 16 and 17 of the Tenth Census are devoted to water powers, and contain the results of detailed studies of the important rivers of the United States. During the Census of 1900 schedules were prepared showing the amount of utilized water power in the United States. These schedules have never been published, but the data may be obtained on application to the Census Bureau.

Reports of the United States Weather Bureau.—The reports of the United States Weather Bureau contain a large amount of data in regard to precipitation, evaporation, and other factors which affect the run-off of streams. A summary of these data is published each year in the Annual Report of the Chief of the Weather Bureau. The Weather Bureau also publishes each month a "Climate and Crop Report," which gives daily temperature, precipitation, and other information for a limited area, usually one State; and the "The Weather Review," which gives under one cover a monthly summary of the data collected in all sections of the country and special articles in regard to various climatic conditions.

Besides collecting these data in regard to climate, the Weather Bureau maintains a flood service, in connection with which a large number of river-height observation stations are maintained for the records of daily fluctuations of stage. These data have been printed under the title "Daily River Stages," of which volumes 1 to 7 have been published.

Reports of the Chief of Engineers, United States Army.—The Army Engineers have made extensive investigations in regard to the flow and slope of many of the larger rivers in the United States, among which are the Mississippi, Missouri, Niagara, and St. Lawrence. Data collected in these investigations are published in the Annual Reports of the Chief of Engineers, United States Army, and in reports of special commissions working under the direction of the Chief of Engineers. The Army Engineers also have a large amount of manuscript data relative to the various streams, which may be obtained on application.

Reports of State officials.—A large amount of valuable information has been collected and published by various States. Among these publications may be mentioned the following:

Reports of the New York State Engineer.

Hydrology of the State of New York, by G. W. Rafter (1905).

Reports of the Illinois State Board of Health.

Report of the Geological Survey of New Jersey, vol. 3 (1894).

Reports of the State Board of Health of Massachusetts.

Reports of the Metropolitan Water and Sewage Board of Massachusetts.

Wells, "Water Powers of Maine" (1869).

Bulletins of the Geological Survey of North Carolina.

Reports of the Commission of Public Works of California.

Reports of special commissions.—A large number of special hydrographic problems have been investigated by commissions appointed by Federal, State, or city governments. Among these reports may be mentioned:

Report of New York City Water Supply, by John R. Freeman (1900).

Report of the Commission on Additional Water Supply of New York City (1904).

Report of Commission on Charles River dam (1903).

Report of the U. S. Deep Waterways Commission (1900).

Report of the State Water Storage Commission of New York (1903).

Reports of city officials.—Nearly all large cities have investigated and reported upon the water supply in their locality in connection with the municipal waterworks. These reports may usually be obtained by application to the city engineer.

How to obtain Government publications.—Most Government publications may be obtained or consulted in the following ways:

(1) A limited number of every issue is delivered to the Department under which the work was done. Copies of these reports may be

obtained either free of charge or for a nominal sum on application to the Department publishing them.

(2) Every member of Congress is allotted a certain number, from which they may be obtained, free of charge, on application.

(3) Other copies are deposited with the Superintendent of Documents, Washington, D. C., from whom they may be purchased practically at cost.

(4) Copies are furnished to the principal public libraries in the large cities throughout the United States, where they may be consulted by those interested.

TABLES.

TABLES.

There are available a large number of tables for facilitating the computations in various hydraulic problems. It is often necessary, however, for the engineer to prepare special tables adapted to the problem in hand. Among the tables available are a number having wide application, which are given on the following pages.

These tables have been adapted from Water-Supply Papers of the U. S. Geological Survey.

In connection with the use of these tables attention is called to Barlow's tables and to Crelle's Rechentafeln. The former tables give for numbers from 1 to 10,000 the squares, cubes, square roots, cube roots, etc. The latter give products of all numbers between 1 and 1000, and can be used both for multiplication and division.

LIST OF TABLES.

Table I.—Discharges in second-feet, over rectangular sharp-crested weirs having

	Head.		Length of weir.											
	Inches.	Feet.	4 in.	6 in.	9 in.	12 in.	15 in.	18 in.	24 in.	2 ft., 6 in.	3 ft.	3 ft., 6 in.	4 ft.	4 ft., 6 in.
1	1/8	0.010	0.0011	0.0017	0.0026	0.00353								
2	1/4	.021	.0033	.0050	.0075	.00997	0.0125	0.0150	0.0200	0.0250	0.0300	0.0350	0.0400	0.0450
3	3/8	.031	.0060	.0091	.0137	.0183	.0229	.0275	.0367	.0459	.0551	.0643	.0735	.0821
4	1/2	.042	.0092	.0139	.0210	.0281	.0352	.0422	.0564	.0706	.0847	.0989	.113	.127
5	5/8	.052	.0128	.0194	.0293	.0392	.0491	.0590	.0788	.0986	.118	.138	.158	.178
6	3/4	.062	.0167	.0254	.0384	.0514	.0644	.0774	.103	.129	.155	.181	.207	.233
7	7/8	.073	.0209	.0318	.0482	.0646	.0810	.0974	.130	.163	.196	.229	.261	.294
8	1	.083	.0254	.0387	.0587	.0788	.0988	.119	.159	.199	.239	.279	.319	.359
9	1 1/8	.094	.0301	.0460	.0699	.0938	.118	.142	.189	.237	.285	.333	.381	.428
10	1 1/4	.104	.0349	.0536	.0816	.110	.138	.166	.222	.278	.334	.389	.445	.501
11	1 3/8	.115	.0401	.0616	.0939	.126	.159	.191	.255	.320	.385	.449	.514	.578
12	1 1/2	.125	.0454	.0699	.107	.144	.180	.217	.291	.364	.438	.511	.585	.659
13	1 5/8	.135	.0508	.0784	.120	.161	.203	.244	.327	.410	.493	.576	.659	.742
14	1 3/4	.146	.0564	.0873	.134	.180	.226	.273	.365	.458	.551	.644	.736	.829
15	1 7/8	.156	.0622	.0965	.148	.199	.251	.302	.405	.508	.611	.714	.816	.919
16	2	.167	.0680	.106	.162	.219	.276	.332	.446	.559	.672	.786	.899	1.012
17	2 1/8	.177	.0740	.115	.177	.239	.301	.364	.488	.612	.736	.860	.984	1.108
18	2 1/4	.187	.0799	.125	.193	.260	.328	.395	.531	.666	.801	.936	1.071	1.206
19	2 3/8	.198	.0862	.135	.208	.282	.355	.428	.575	.722	.868	1.015	1.161	1.308
20	2 1/2	.208	.0924	.145	.224	.303	.383	.462	.620	.778	.937	1.095	1.253	1.412
21	2 5/8	.219		.155	.241	.326	.411	.496	.667	.837	1.007	1.178	1.348	1.518
22	2 3/4	.229		.166	.257	.349	.440	.531	.714	.897	1.079	1.262	1.444	1.627
23	2 7/8	.240		.177	.274	.372	.469	.567	.762	.958	1.153	1.348	1.543	1.739
24	3	.250		.187	.291	.395	.499	.604	.812	1.020	1.228	1.436	1.644	1.852
25	3 1/8	.260		.198	.309	.420	.530	.641	.862	1.083	1.305	1.526	1.747	1.968
26	3 1/4	.271		.209	.327	.444	.561	.679	.913	1.148	1.383	1.617	1.852	2.087
27	3 3/8	.281		.220	.345	.469	.593	.717	.966	1.214	1.462	1.711	1.959	2.207
28	3 1/2	.292		.232	.363	.494	.625	.756	1.018	1.281	1.543	1.805	2.068	2.330
29	3 5/8	.302			.381	.520	.658	.796	1.072	1.349	1.625	1.902	2.178	2.455
30	3 3/4	.312			.400	.545	.691	.836	1.127	1.418	1.709	2.000	2.291	2.581
31	3 7/8	.323			.419	.572	.724	.877	1.183	1.488	1.794	2.099	2.405	2.710
32	4	.333			.438	.598	.758	.919	1.239	1.559	1.880	2.200	2.521	2.841
33	4 1/8	.344			.457	.625	.793	.961	1.296	1.632	1.967	2.303	2.638	2.974
34	4 1/4	.354			.477	.652	.828	1.003	1.354	1.705	2.056	2.407	2.758	3.109
35	4 3/8	.365			.496	.680	.863	1.046	1.413	1.779	2.146	2.512	2.879	3.245
36	4 1/2	.375			.516	.707	.899	1.090	1.472	1.854	2.237	2.619	3.001	3.384
37	4 5/8	.385				.735	.935	1.134	1.532	1.930	2.329	2.727	3.126	3.524
38	4 3/4	.396				.764	.971	1.178	1.593	2.008	2.422	2.837	3.252	3.666
39	4 7/8	.406				.793	1.009	1.224	1.655	2.086	2.517	2.949	3.380	3.811
40	5	.417				.821	1.045	1.269	1.717	2.164	2.612	3.060	3.508	3.956
41	5 1/8	.427				.850	1.082	1.315	1.779	2.244	2.709	3.174	3.638	4.103
42	5 1/4	.437				.879	1.120	1.361	1.843	2.324	2.806	3.288	3.770	4.252
43	5 3/8	.448				.908	1.158	1.407	1.906	2.405	2.904	3.404	3.903	4.402
44	5 1/2	.458				.938	1.196	1.455	1.971	2.488	3.005	3.521	4.038	4.555
45	5 5/8	.469				.968	1.235	1.502	2.037	2.571	3.105	3.640	4.174	4.708
46	5 3/4	.479				.998	1.274	1.550	2.103	3.255	3.207	3.760	4.312	4.864
47	5 7/8	.490				1.029	1.314	1.599	2.170	2.740	3.310	3.881	4.451	5.021
48	6	.500				1.060	1.354	1.649	2.237	2.826	3.415	4.003	4.592	5.181
49	6 1/8	.510					1.394	1.697	2.304	2.912	3.519	4.126	4.733	5.540
50	6 1/4	.521					1.434	1.747	2.372	2.999	3.624	4.250	4.876	5.302
51	6 3/8	.531					1.475	1.798	2.442	3.087	3.732	4.377	5.021	5.666
52	6 1/2	.542					1.516	1.848	2.511	3.175	3.839	4.503	5.166	5.830
53	6 5/8	.552					1.557	1.898	2.581	3.264	3.947	4.630	5.313	5.996
54	6 3/4	.563					1.598	1.949	2.652	3.354	4.057	4.759	5.461	6.164
55	6 7/8	.573					1.640	2.001	2.723	3.445	4.167	4.889	5.611	6.333
56	7	.583					1.682	2.053	2.795	3.536	4.278	5.020	5.762	6.504
57	7 1/8	.594					1.725	2.106	2.867	3.629	4.391	5.153	5.914	6.676
58	7 1/4	.604					1.767	2.158	2.940	3.722	4.504	5.286	6.067	6.849
59	7 3/8	.615					1.809	2.210	3.012	3.815	4.617	5.419	6.221	7.023
60	7 1/2	.625					1.851	2.263	3.085	3.908	4.731	5.554	6.376	7.199
61	7 5/8	.635						2.316	3.160	4.003	4.846	5.690	6.533	7.376
62	7 3/4	.646						2.370	3.234	4.098	4.963	5.827	6.691	7.555
63	7 7/8	.656						2.423	3.308	4.193	5.079	5.964	6.849	7.734
64	8	.667						2.477	3.384	4.290	5.196	6.103	7.009	7.915
65	8 1/4	.688						2.586	3.535	4.484	5.433	6.383	7.332	8.281
66	8 1/2	.708						2.697	3.689	4.682	5.674	6.667	7.660	8.652
67	8 3/4	.729						2.808	3.844	4.881	5.918	6.955	7.991	9.028
68	9	.750						2.919	4.001	5.082	6.164	7.245	8.327	9.408
69	9 1/2	.792							4.320	5.492	6.665	7.838	9.011	10.184
70	10	.833							4.644	5.911	7.177	8.444	9.711	10.977

complete end contractions. [*Formula* $Q = 3.33\ (l - .2H)\ H^{\frac{3}{2}}$].

Length of weir—Continued.											Addition for increase of length.		
5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.	12 ft.	14 ft.	16 ft	18 ft.	20 ft.	1 in.	1 ft.	
......	......	......	......	......	......	......	......	......	......	......	0.00029	0.00354	1
0.0500	0.0600	0.0700	0.0800	0.0900	0.100	0.120	0.140	0.160	0.180	0.200	.00083	.01001	2
.0918	.110	.129	.147	.166	.184	.221	.258	.294	.331	.368	.00153	.0184	3
.141	.170	.198	.226	.255	.283	.340	.396	.453	.510	.566	.00236	.0283	4
.198	.237	.277	.316	.356	.395	.475	.554	.633	.712	.791	.00330	.0396	5
.260	.312	.364	.416	.468	.520	.624	.728	.832	.936	1.040	.00433	.0520	6
.327	.392	.458	.524	.589	.655	.786	.917	1.048	1.179	1.310	.00546	.0656	7
.399	.479	.559	.640	.720	.800	.960	1.120	1.280	1.441	1.601	.00667	.0801	8
.476	.572	.667	.763	.859	.954	1.145	1.337	1.528	1.719	1.910	.00796	.0956	9
.557	.669	.781	.893	1.005	1.117	1.341	1.565	1.789	2.013	2.237	.00933	.112	10
.643	.772	.901	1.030	1.160	1.289	1.547	1.805	2.064	2.322	2.580	.0108	.129	11
.732	.879	1.026	1.174	1.321	1.468	1.762	2.057	2.351	2.645	2.940	.0123	.147	12
.825	.991	1.157	1.323	1.489	1.655	1.967	2.319	2.650	2.982	3.314	.0138	.166	13
.922	1.107	1.293	1.478	1.664	1.849	2.220	2.591	2.962	3.333	3.704	.0155	.185	14
1.022	1.228	1.433	1.639	1.845	2.050	2.462	2.873	3.284	3.696	4.107	.0171	.206	15
1.125	1.352	1.579	1.805	2.032	2.258	2.711	3.165	3.618	4.071	4.524	.0189	.227	16
1.232	1.480	1.728	1.976	2.225	2.473	2.969	3.465	3.962	4.458	4.954	.0207	.248	17
1.342	1.612	1.882	2.153	2.423	2.693	3.234	3.775	4.316	4.856	5.397	.0225	.270	18
1.455	1.748	2.041	2.334	2.627	2.921	3.507	4.093	4.680	5.266	5.853	.0244	.293	19
1.570	1.887	2.203	2.520	2.837	3.153	3.787	4.420	5.053	5.686	6.320	.0264	.317	20
1.689	2.029	2.370	2.711	3.051	3.392	4.074	4.755	5.436	6.118	6.799	.0284	.341	21
1.810	2.175	2.540	2.906	3.271	3.636	4.367	5.098	5.828	6.559	7.290	.0304	.365	22
1.934	2.324	2.715	3.105	3.496	3.886	4.667	5.448	6.229	7.010	7.792	.0325	.391	23
2.060	2.477	2.893	3.309	3.725	4.142	4.974	5.807	6.639	7.472	8.304	.0347	.416	24
2.190	2.632	3.075	3.517	3.960	4.402	5.287	6.172	7.058	7.943	8.828	.0369	.443	25
2.321	2.791	3.260	3.729	4.199	4.668	5.607	6.546	7.484	8.423	9.362	.0391	.469	26
2.455	2.952	3.449	3.946	4.442	4.939	5.932	6.926	7.919	8.913	9.906	.0414	.497	27
2.592	3.117	3.641	4.166	4.690	5.215	6.264	7.313	8.362	9.411	10.460	.0437	.525	28
2.731	3.284	3.837	4.390	4.943	5.495	6.601	7.707	8.813	9.918	11.024	.0461	.553	29
2.872	3.454	4.036	4.617	5.199	5.781	6.944	8.108	9.271	10.435	11.598	.0485	.582	30
3.016	3.627	4.238	4.849	5.460	6.071	7.293	8.515	9.737	10.960	12.182	.0509	.611	31
3.162	3.802	4.443	5.084	5.725	6.366	7.648	8.929	10.211	11.493	12.774	.0534	.641	32
3.310	3.981	4.652	5.323	5.994	6.665	8.007	9.350	10.692	12.034	13.376	.0559	.671	33
3.460	4.161	4.863	5.565	6.267	6.969	8.373	9.776	11.180	12.584	13.988	.0585	.702	34
3.612	4.345	5.078	5.811	6.544	7.277	8.743	10.209	11.676	13.142	14.608	.0611	.733	35
3.766	4.531	5.296	6.060	6.825	7.590	9.119	10.648	12.178	13.707	15.237	.0637	.765	36
3.922	4.719	5.516	6.313	7.110	7.906	9.500	11.093	12.687	14.281	15.874	.0664	.797	37
4.081	4.910	5.739	6.569	7.398	8.227	9.886	11.545	13.203	14.862	16.520	.0691	.829	38
4.242	5.104	5.967	6.829	7.691	8.553	10.278	12.002	13.727	15.451	17.176	.0719	.862	39
4.404	5.299	6.195	7.090	7.986	8.882	10.673	12.464	14.256	16.047	17.838	.0747	.896	40
4.568	5.497	6.427	7.356	8.285	9.215	11.074	12.933	14.791	16.650	18.509	.0775	.929	41
4.733	5.697	6.661	7.624	8.588	9.551	11.479	13.406	15.333	17.260	19.187	.0803	.964	42
4.901	5.899	6.897	7.895	8.894	9.892	11.888	13.885	15.881	17.877	19.874	.0832	.998	43
5.071	6.105	7.138	8.171	9.204	10.238	12.304	14.371	16.438	18.504	20.571	.0861	1.033	44
5.243	6.312	7.380	8.449	9.518	10.586	12.724	14.861	16.999	19.136	21.273	.0891	1.069	45
5.416	6.521	7.626	8.730	9.835	10.939	13.149	15.358	17.567	19.776	21.985	.0921	1.105	46
5.592	6.733	7.873	9.014	10.155	11.295	13.577	15.858	18.140	20.421	22.702	.0951	1.141	47
5.769	6.947	8.124	9.301	10.478	11.656	14.010	16.365	18.720	21.074	23.429	.0981	1.177	48
5.948	7.162	8.376	9.591	10.805	12.020	14.448	16.877	19.306	21.735	24.164	.101	1.214	49
6.128	7.380	8.631	9.883	11.135	12.386	14.890	17.393	19.897	22.400	24.903	.104	1.252	50
6.311	7.600	8.889	10.179	11.468	12.758	15.336	17.915	20.494	23.073	25.652	.107	1.289	51
6.494	7.822	9.149	10.477	11.804	13.132	15.787	18.442	21.097	23.752	26.407	.111	1.328	52
6.679	8.045	9.411	10.777	12.143	13.509	16.241	18.973	21.705	24.437	27.169	.114	1.366	53
6.866	8.271	9.676	11.081	12.485	13.890	16.700	19.509	22.319	25.129	27.938	.117	1.405	54
7.055	8.500	9.944	11.388	12.832	14.276	17.164	20.052	22.941	25.829	28.717	.120	1.444	55
7.245	8.729	10.213	11.696	13.180	14.663	17.631	20.598	23.565	26.532	29.499	.124	1.484	56
7.438	8.962	10.485	12.009	13.532	15.056	18.103	21.150	24.197	27.244	30.291	.127	1.524	57
7.631	9.195	10.759	12.323	13.886	15.450	18.578	21.705	24.833	27.961	31.088	.130	1.564	58
7.826	9.430	11.034	12.639	14.243	15.848	19.056	22.265	25.474	28.683	31.892	.134	1.604	59
8.022	9.667	11.312	12.958	14.603	16.249	19.539	22.830	26.121	29.412	32.703	.137	1.645	60
8.220	9.907	11.593	13.280	14.967	16.653	20.027	23.400	26.774	30.147	33.520	.141	1.687	61
8.419	10.148	11.876	13.604	15.332	17.061	20.517	23.974	27.431	30.887	34.344	.144	1.728	62
8.619	10.390	12.160	13.930	15.700	17.471	21.011	24.552	28.093	31.633	35.174	.148	1.770	63
8.821	10.634	12.447	14.259	16.072	17.884	21.510	25.135	28.760	32.385	36.010	.151	1.813	64
9.230	11.128	13.026	14.924	16.823	18.721	22.517	26.314	30.110	33.906	37.703	.158	1.898	65
9.645	11.630	13.615	15.600	17.586	19.571	23.541	27.512	31.482	35.452	39.423	.165	1.985	66
10.065	12.138	14.211	16.285	18.358	20.432	24.578	28.725	32.872	37.019	41.166	.173	2.073	67
10.490	12.653	14.815	16.978	19.141	21.304	25.630	29.956	34.282	38.607	42.933	.180	2.163	68
11.356	13.702	16.048	18.393	20.739	23.084	27.776	32.467	37.158	41.849	46.540	.196	2.346	69
12.244	14.777	17.310	19.843	22.377	24.910	29.976	35.043	40.109	45.175	50.242	.211	2.533	70

Table II.—Discharge in second-feet per foot of crest over rectangular sharp-crested weirs without end contractions.

Formula: $Q=3.33\, l\, H^{\frac{3}{2}}$.

Head H, feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.0000	0.0333	0.0094	0.0173	0.0266	0.0372	0.0489	0.0617	0.0753	0.0899
.1	.1053	.1215	.1384	.1561	.1744	.1935	.2131	.2334	.2543	.2758
.2	.2978	.3205	.3436	.3673	.3915	.4162	.4415	.4672	.4934	.5200
.3	.5472	.5748	.6028	.6313	.6602	.6895	.7193	.7495	.7800	.8110
.4	.8424	.8742	.9064	.9390	.9719	1.0052	1.0389	1.0730	1.1074	1.1422
.5	1.1773	1.2128	1.2487	1.2849	1.3214	1.3583	1.3955	1.4330	1.4709	1.5091
.6	1.5476	1.5865	1.6257	1.6652	1.7050	1.7451	1.7856	1.8262	1.8673	1.9086
.7	1.9503	1.9922	2.0344	2.0770	2.1198	2.1629	2.2063	2.2500	2.2940	2.3382
.8	2.3828	2.4276	2.4727	2.5180	2.5637	2.6096	2.6558	2.7022	2.7490	2.7959
.9	2.8432	2.8907	2.9385	2.9865	3.0348	3.0834	3.1322	3.1813	3.2306	3.2802
1.0	3.3300	3.3801	3.4304	3.4810	3.5318	3.5828	3.6342	3.6857	3.7375	3.7895
1.1	3.8418	3.8943	3.9470	4.0000	4.0532	4.1067	4.1604	4.2143	4.2384	4.3228
1.2	4.3774	4.4322	4.4873	4.5426	4.5981	4.6538	4.7098	4.7660	4.8224	4.8790
1.3	4.9358	4.9929	5.0502	5.1077	5.1654	5.2233	5.2814	5.3398	5.3984	5.4572
1.4	5.5162	5.5754	5.6348	5.6944	5.7542	5.8143	5.8745	5.9350	5.9957	6.0565
1.5	6.1176	6.1789	6.2404	6.3020	6.3638	6.4260	6.4883	6.5508	6.6135	6.6764
1.6	6.7394	6.8027	6.8662	6.9299	6.9937	7.0578	7.1221	7.1865	7.2512	7.3160
1.7	7.3810	7.4463	7.5117	7.5773	7.6431	7.7091	7.7752	7.8416	7.9081	7.9749
1.8	8.0418	8.1089	8.1762	8.2437	8.3113	8.3792	8.4472	8.5154	8.5838	8.6524
1.9	8.7212	8.7901	8.8592	8.9285	8.9980	9.0677	9.1375	9.2075	8.2777	9.3481
2.0	9.4187	9.4894	9.5603	9.6314	9.7026	9.7741	9.8457	9.9174	9.9894	10.0620
2.1	10.1340	10.2060	10.2790	10.3520	10.4250	10.4980	10.5710	10.6450	10.7180	10.7920
2.2	10.8660	10.9400	11.0150	11.0900	11.1640	11.2390	11.3140	11.3890	11.4640	11.5400
2.3	11.6150	11.6910	11.7670	11.8430	11.9200	11.9960	12.0730	12.1500	12.2270	12.3040
2.4	12.3810	12.4590	12.5360	12.6140	12.6920	12.7700	12.8480	12.9270	13.0050	13.0840
2.5	13.1630	13.2480	13.3210	13.4010	13.4800	13.5600	12.6400	13.7200	13.8000	13.8800
2.6	13.9610	14.0410	14.1220	14.2030	14.2840	14.3650	14.4470	14.5280	14.6100	14.6920
2.7	14.7740	14.8560	14.9380	15.0210	15.1030	15.1860	15.2690	15.3520	15.4350	15.5190
2.8	15.6020	15.6860	15.7690	15.8530	15.9380	16.0220	16.1060	16.1910	16.2750	16.3600
2.9	16.4450	16.5300	16.6160	16.7010	16.7870	16.8720	16.9580	17.0440	17.1300	17.2170
3.0	17.3033	17.3899	17.4698	17.5634	17.6503	17.7376	17.8248	17.9124	18.0000	18.0876
3.1	18.1754	18.2634	18.3516	18.4399	18.5285	18.6170	18.7056	18.7945	18.8838	18.9727
3.2	19.0619	19.1515	19.2410	19.3307	19.4206	19.5106	19.6007	19.6910	19.7812	19.8718
3.3	19.9624	20.0528	20.1442	20.2354	20.3267	20.4179	20.5095	20.6011	20.6930	20.7849
3.4	20.8777	20.9690	21.0613	21.1538	21.2464	21.3390	21.4319	21.5248	21.6180	21.7113
3.5	21.8045	21.8980	21.9917	22.0856	22.1795	22.2734	22.3677	22.4618	22.5564	22.6510
3.6	22.7456	22.8405	22.9354	23.0306	23.1259	23.2211	23.3167	23.4122	23.5081	23.6040
3.7	23.6999	23.7962	23.8924	23.9887	24.0852	24.1818	24.2787	24.3756	24.4728	24.5697
3.8	24.6673	24.7645	24.8621	24.9600	25.0576	25.1555	25.2537	25.3520	25.4502	25.5488
3.9	25.6473	25.7459	25.8748	25.9437	26.0429	26.1422	26.2414	26.3410	26.4405	26.5401
4.0	26.6400	26.7399	26.8401	26.9404	27.0406	27.1412	27.2417	27.3423	27.4432	27.5411
4.1	27.6453	27.7466	27.8478	27.9494	28.0509	28.1525	28.2544	28.3563	28.4582	28.5604
4.2	28.6626	28.7652	28.8678	28.9703	29.0732	29.1761	29.2790	29.3823	29.4855	29.5890
4.3	29.6926	29.7962	29.9001	30.0040	30.1079	30.2118	30.3163	30.4205	30.5251	30.6297
4.4	30.7342	30.8391	30.9440	31.0493	31.1545	31.2597	31.3649	31.4705	31.5764	31.6820
4.5	31.7878	31.8941	32.0003	32.1065	32.2128	32.3193	32.4259	32.5324	32.6393	32.7462
4.6	32.8534	32.9607	33.0679	33.1755	33.2830	33.3906	33.4985	33.6064	33.7143	33.8225
4.7	33.9307	34.0373	34.1475	34.2560	34.3646	34.4735	34.5824	34.6913	34.8005	34.9097
4.8	35.0193	35.1288	35.2354	35.3480	35.4578	35.5677	35.6780	35.7882	35.8984	36.0086
4.9	36.1182	36.2297	36.3406	36.4515	36.5624	36.6736	36.7845	36.8961	37.0073	37.1188

Table II.—Continued.

Head *H*, feet.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
* 5.0	37.2304	37.3423	37.4542	37.5661	37.6783	37.7905	37.9027	38.0153	38.1275	38.2404
5.1	38.3529	38.4658	38.5787	38.6919	38.8052	38.9184	39.0319	39.1455	39.2591	39.3726
5.2	39.4865	39.6004	39.7146	39.8288	39.9430	40.0576	40.1718	40.2867	40.4012	40.5161
5.3	40.6310	40.7462	40.5281	40.9766	41.0919	41.2074	41.3230	41.4386	41.5544	41.6703
5.4	41.7866	41.9024	42.0186	42.1352	42.2517	42.3683	42.4848	42.6017	42.7186	42.8355
5.5	42.9523	43.0700	43.1871	43.3043	43.4219	42.5394	43.6573	43.7752	43.8931	44.0109
5.6	44.1292	44.2474	44.3659	44.4845	44.6030	44.7216	44.8404	44.9593	45.0782	45.1974
5.7	45.3166	45.4359	45.5554	45.6746	45.7945	45.9140	46.0339	46.1538	46.2740	46.3939
5.8	46.5141	46.6347	46.7552	46.8757	46.9963	47.1172	47.2380	47.3589	47.4798	47.6010
5.9	47.7226	47.8438	47.9653	48.0869	48.2084	48.3303	48.4522	48.5744	48.6963	48.8185
6.0	48.9407	49.0632	49.1858	49.3083	48.4312	49.5537	49.6766	49.7999	49.9230	50.0462
6.1	50.1694	50.2930	50.4162	50.5401	50.6637	50.7875	50.9114	51.0356	51.1595	51.2837
6.2	51.4082	51.5324	51.6570	51.7818	51.9034	52.0313	52.1531	52.2813	52.4062	52.5314
6.3	52.6570	52.7822	52.9077	53.0336	53.1591	53.2850	53.4109	53.5371	53.6630	53.7892
6.4	53.9157	54.0419	54.1684	54.2950	54.4219	54.5487	54.6756	54.8025	54.9297	55.0569
6.5	55.1832	55.3116	55.4392	55.5667	55.6943	55.8221	55.9500	56.0779	56.2001	56.3343
6.6	56.4625	56.5910	56.7192	56.8478	56.9766	57.1055	57.2340	57.3233	57.4921	57.6213
6.7	57.7505	57.8801	58.0093	58.1388	58.2687	58.3982	58.5281	58.6580	58.7882	58.9180
6.8	59.0482	59.1788	59.3090	59.4428	59.5700	59.7009	59.8314	59.9623	60.0935	60.2244
6.9	60.3556	60.4868	60.6183	60.7499	60.8814	61.0129	61.1445	61.2763	61.4082	61.5404
7.0	61.6736	61.8048	61.9370	62.0692	62.2017	62.3343	62.4671	62.6000	62.7329	62.8657
7.1	62.9986	63.1318	63.2650	63.3992	63.5317	63.6653	63.7991	63.9327	64.0665	64.2004
7.2	64.3343	64.4685	64.6027	64.7369	64.8711	65.0056	65.1268	65.2750	65.4095	65.5444
7.3	65.6793	65.8145	65.9493	66.0845	66.2197	66.3552	66.4908	66.6263	66.7618	66.8977
7.4	67.0336	67.1694	67.3053	67.4415	67.5777	67.7139	67.8504	67.9869	68.1235	68.2600
7.5	68.3969	68.5337	68.6706	68.8078	68.9447	69.0818	69.2794	69.3566	69.4941	69.6316
7.6	69.7695	69.9070	70.0449	70.1827	70.3209	70.4591	70.5973	70.7355	70.8737	71.0123
7.7	71.1508	71.2896	71.4282	71.5670	71.7059	71.8451	71.9843	72.1235	72.2627	72.4743
7.8	72.5414	72.6809	72.8208	72.9603	73.1002	73.2400	73.3802	73.5201	73.6603	73.8005
7.9	73.9410	74.0815	74.2220	74.3626	74.5031	74.6439	74.7848	74.9260	75.0669	75.2081
8.0	75.3492	75.4908	75.6320	75.7735	75.9150	76.0569	76.1987	76.3406	76.4824	76.6243
8.1	76.7665	76.9087	77.0509	77.1934	77.3360	77.4784	77.6210	77.7638	77.9067	78.0496
8.2	78.1924	78.3356	78.4788	78.6220	78.7655	78.9087	79.0522	79.1957	79.3396	79.4834
8.3	79.6273	79.7711	79.9153	80.0592	80.2034	80.3479	80.4921	80.6366	80.7811	80.9260
8.4	81.0705	81.2154	81.3602	81.5054	81.6503	81.7955	81.9406	82.0862	82.2314	82.3769
8.5	82.5224	82.6682	82.8141	82.9600	83.1058	83.2517	83.3979	83.5440	83.6902	83.8367
8.6	83.9833	84.1298	84.2763	84.4228	84.5697	84.7165	84.8634	85.0106	85.1578	85.3049
8.7	85.4521	85.5996	85.7472	85.8947	86.0455	86.1897	86.3376	86.4851	86.6336	86.7815
8.8	86.9297	87.0778	87.2264	87.3745	87.5231	87.6716	87.8204	87.9689	88.1178	88.2666
8.9	88.4192	88.5647	88.7139	88.8630	89.0126	89.1617	89.3113	89.4608	89.6103	89.7602
9.0	89.9100	90.0599	90.2064	90.3599	90.5101	90.6602	90.4778	90.9609	91.1115	91.2620
9.1	91.4125	91.5633	91.7142	91.8650	92.0159	92.1671	92.3183	92.4694	92.6206	92.7721
9.2	92.9237	93.0782	93.2267	93.3785	93.5304	93.6822	93.8341	93.9863	94.1384	94.2906
9.3	94.4428	94.5950	94.7475	94.9000	95.0529	95.2054	95.3582	95.5111	95.6639	95.8171
9.4	95.9703	96.1234	96.2766	96.4298	96.5833	96.7368	96.8903	97.0442	97.1977	97.3516
9.5	97.5057	97.6596	97.8138	97.9679	98.1021	98.2763	98.4308	98.5853	98.7398	98.8943
9.6	99.0492	99.2040	99.3589	99.5141	99.6689	99.8241	99.9793	100.1344	100.2899	100.4455
9.7	100.6010	100.7565	100.9123	101.0678	101.2237	101.3799	101.5357	101.6919	101.8481	102.0042
9.8	102.1607	102.3169	102.4734	102.6299	102.7868	102.9433	103.1001	103.2570	103.4141	103.5710
9.9	103.7282	103.8853	104.0429	104.2000	104.3575	104.5121	104.6726	104.8304	104.9882	105.1461
10.0	105.3039	105.4618	105.6199	105.7781	105.9363	106.0945	106.2530	106.4115	106.5700	106.7285

NOTE:—By increasing the quantities in this table by 1 per cent the discharge by the Cippoletti formula ($Q=3.3^{7}_{3}\ lH^{\frac{3}{2}}$) will be obtained.

Table III.—Discharge in second-feet per foot of crest length for certain broad-crested weirs.

Formula: $Q=2.64\ lH^{\frac{3}{2}}$

Head H, feet	0	1	2	3	4	5	6	7	8	9	10
0.00	0.000	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5
.01	.003	2.68	7.52	13.8	21.2	29.6	38.9	49.0	59.8	71.4	83.6
.02	.007	2.72	7.58	13.8	21.3	29.7	39.0	49.1	59.9	71.5	83.7
.03	.014	2.76	7.64	13.9	21.4	29.8	39.1	49.2	60.1	71.6	83.9
.04	.021	2.80	7.69	14.0	21.4	29.9	39.2	49.3	60.2	71.7	84.0
.05	.030	2.84	7.75	14.1	21.5	30.0	39.3	49.4	60.3	71.9	84.1
.06	.039	2.88	7.81	14.1	21.6	30.0	39.4	49.5	60.4	72.0	84.2
.07	.049	2.92	7.86	14.2	21.7	30.1	39.5	49.6	60.5	72.1	84.4
.08	.060	2.96	7.92	14.3	21.8	30.2	39.6	49.7	60.6	72.2	84.5
.09	.071	3.00	7.98	14.3	21.8	30.3	39.7	49.8	60.7	72.3	84.6
0.10	0.083	3.04	8.03	14.4	21.9	30.4	39.8	49.9	60.8	72.5	84.7
.11	.096	3.09	8.09	14.5	22.0	30.5	39.9	50.0	61.0	72.6	84.9
.12	.110	3.13	8.15	14.5	22.1	30.6	40.0	50.2	61.1	72.7	85.0
.13	.124	3.17	8.21	14.6	22.2	30.7	40.1	50.3	61.2	72.8	85.1
.14	.138	3.21	8.26	14.7	22.2	30.8	40.2	50.4	61.3	72.9	85.2
.15	.153	3.26	8.32	14.8	22.3	30.8	40.3	50.5	61.4	73.1	85.4
.16	.169	3.30	8.38	14.8	22.4	30.9	40.4	50.6	61.5	73.2	85.5
.17	.185	3.34	8.44	14.9	22.5	31.0	40.5	50.7	61.6	73.3	85.6
.18	.202	3.38	8.50	15.0	22.6	31.1	40.6	50.8	61.8	73.4	85.7
.19	.218	3.43	8.56	15.0	22.6	31.2	40.7	50.9	61.9	73.5	85.9
0.20	0.236	3.47	8.61	15.1	22.7	31.3	40.8	51.0	62.0	73.7	86.0
.21	.254	3.51	8.67	15.2	22.8	31.4	40.9	51.1	62.1	73.8	86.1
.22	.272	3.56	8.73	15.2	22.9	31.5	41.0	51.2	62.2	73.9	86.2
.23	.291	3.60	8.79	15.3	23.0	31.6	41.0	51.3	62.3	74.0	86.4
.24	.310	3.64	8.85	15.4	23.0	31.7	41.1	51.4	62.4	74.1	86.5
.25	.330	3.69	8.91	16.5	23.1	31.8	41.2	51.5	62.6	74.3	86.6
.26	.350	3.73	8.97	15.5	23.2	31.8	41.3	51.6	62.7	74.4	86.8
.27	.370	3.78	9.03	15.6	23.3	31.9	41.4	51.7	62.8	74.5	86.9
.28	.391	3.82	9.09	15.7	23.4	32.0	41.5	51.9	62.9	74.6	87.0
.29	.412	3.87	9.15	15.8	23.4	32.1	41.6	52.0	63.0	74.8	87.1
0.30	0.434	3.91	9.21	15.8	23.5	32.2	41.7	52.1	63.1	74.9	87.3
.31	.456	3.96	9.27	15.9	23.6	32.3	41.8	52.2	63.2	75.0	87.4
.32	.478	4.00	9.33	16.0	23.7	32.4	41.9	52.3	63.4	75.1	87.5
.33	.500	4.05	9.39	16.0	23.8	32.5	42.0	52.4	63.5	75.2	87.6
.34	.524	4.10	9.45	16.2	23.9	32.6	42.1	52.5	63.6	75.4	87.8
.35	.547	4.14	9.51	16.2	24.0	32.7	42.2	52.6	63.7	75.5	87.9
.36	.570	4.19	9.57	16.3	24.1	32.8	42.3	52.7	63.8	75.6	88.0
.37	.594	4.23	9.63	16.3	24.1	32.8	42.4	52.8	63.9	75.7	88.2
.38	.618	4.28	9.69	16.4	24.2	32.9	42.5	52.9	64.0	75.8	88.3
.39	.643	4.33	9.75	16.5	24.3	33.0	42.6	53.0	64.2	76.0	88.4
0.40	0.668	4.37	9.82	16.6	24.4	33.1	42.7	53.1	64.3	76.1	88.5
.41	.693	4.42	9.88	16.6	24.4	33.2	42.8	53.2	64.4	76.2	88.7
.42	.719	4.47	9.94	16.7	24.5	33.3	42.9	53.4	64.5	76.3	88.8
.43	.744	4.51	10.0	16.8	24.6	33.4	43.0	53.5	64.6	76.4	88.9
.44	.771	4.56	10.1	16.8	24.7	33.5	43.1	53.6	64.7	76.6	89.0
.45	.797	4.61	10.1	16.9	24.8	33.6	43.2	53.7	64.8	76.7	89.2
.46	.824	4.66	10.2	17.0	24.9	33.7	43.3	53.8	65.0	76.8	89.3
.47	.851	4.70	10.2	17.1	24.9	33.8	43.4	53.9	65.1	76.9	89.4
.48	.878	4.75	10.3	17.1	25.0	33.9	43.6	54.0	65.2	77.0	89.6
.49	.905	4.80	10.4	17.2	25.1	34.0	43.6	54.1	65.3	77.2	89.7

Table III.—Continued.

Head *H*, feet.	0	1	2	3	4	5	6	7	8	9	10
0.50	0.934	4.85	10.4	17.3	25.2	34.0	43.7	54.2	65.4	77.3	89.8
.51	.961	4.90	10.5	17.4	25.3	34.1	43.8	54.3	65.5	77.4	90.0
.52	.990	4.95	10.6	17.4	25.4	34.2	44.0	54.4	65.6	77.5	90.1
.53	1.02	5.00	10.6	17.5	25.4	34.3	44.1	54.6	65.8	77.7	90.2
.54	1.05	5.04	10.7	17.6	25.5	34.4	44.2	54.7	65.9	77.8	90.3
.55	1.08	5.09	10.8	17.7	25.6	34.5	44.3	54.8	66.0	77.9	90.5
.56	1.11	5.14	10.8	17.7	25.7	34.6	44.4	54.9	66.1	78.0	90.6
.57	1.14	5.19	10.9	17.8	25.8	34.7	44.5	55.0	66.2	78.2	90.7
.58	1.17	5.24	10.9	17.9	25.9	34.8	44.6	55.1	66.3	78.3	90.8
.59	1.20	5.29	11.0	18.0	26.0	34.9	44.7	55.2	66.5	73.4	91.0
0.60	1.23	5.34	11.1	18.0	26.0	35.0	44.8	55.3	66.6	78.5	91.1
.61	1.26	5.39	11.1	18.1	26.1	35.1	44.9	55.4	66.7	78.6	91.2
.62	1.29	5.44	11.2	18.2	26.2	35.2	45.0	55.5	66.8	78.8	91.4
.63	1.32	5.49	11.2	18.2	26.3	35.3	45.1	55.6	66.9	78.9	91.5
.64	1.35	5.54	11.3	18.3	26.4	35.4	45.2	55.7	67.0	79.0	91.6
.65	1.38	5.60	11.4	18.4	26.5	35.4	45.3	55.9	67.2	79.1	91.8
.66	1.42	5.65	11.4	18.5	26.6	35.5	45.4	56.0	67.3	79.3	91.9
.67	1.45	5.70	11.5	18.6	26.6	35.6	45.5	56.1	67.4	79.4	92.0
.68	1.48	5.75	11.6	18.6	26.7	35.7	45.6	56.2	67.5	79.5	92.1
.69	1.51	5.80	11.6	18.7	26.8	35.8	45.7	56.3	67.6	79.6	92.3
0.70	1.55	5.85	11.7	18.8	26.9	35.9	45.8	56.4	67.7	79.8	92.4
.71	1.58	5.90	11.8	18.9	27.0	36.0	45.9	56.5	67.9	79.9	92.5
.72	1.61	5.96	11.8	18.9	27.1	36.1	46.0	56.6	68.0	80.0	92.7
.73	1.65	6.01	11.9	19.0	27.2	36.2	46.1	56.7	68.1	80.1	92.8
.74	1.68	6.06	12.0	19.1	27.2	36.3	46.2	56.8	68.2	80.2	92.9
.75	1.71	6.11	12.0	19.2	27.3	36.4	46.3	57.0	68.3	80.4	93.0
.76	1.75	6.16	12.1	19.2	27.4	36.5	46.4	57.1	68.4	80.5	93.2
.77	1.78	6.22	12.2	19.3	27.5	36.6	46.5	57.2	68.6	80.6	93.3
.78	1.82	6.27	12.2	19.4	27.6	36.7	46.6	57.3	68.7	80.7	93.4
.79	1.85	6.32	12.3	19.5	27.7	36.8	46.7	57.4	68.8	80.9	93.6
0.80	1.89	6.38	12.4	19.6	27.8	36.9	46.8	57.5	68.9	81.0	93.7
.81	1.92	6.43	12.4	19.6	27.8	37.0	46.9	57.6	69.0	81.1	93.8
.82	1.96	6.48	12.5	19.7	27.9	37.1	47.0	57.7	69.2	81.2	94.0
.83	2.00	6.54	12.6	19.8	28.0	37.2	47.1	57.8	69.3	81.4	94.1
.84	2.03	6.59	12.6	19.9	28.1	37.3	47.2	58.0	69.4	81.5	94.2
.85	2.07	6.64	12.7	19.9	28.2	37.4	47.3	58.1	69.5	81.6	94.4
.86	2.10	6.70	12.8	20.0	28.3	37.4	47.4	58.2	69.6	81.7	94.5
.87	2.14	6.75	12.8	20.1	28.4	37.5	47.5	58.3	69.7	81.9	94.6
.88	2.18	6.80	12.9	20.2	28.5	37.6	47.6	58.4	69.9	82.0	94.7
.89	2.22	6.86	13.0	20.2	28.5	37.7	47.7	58.5	70.0	82.1	94.9
0.90	2.25	6.91	13.0	20.3	28.6	37.8	47.8	58.6	70.1	82.2	95.0
.91	2.29	6.97	13.1	20.4	28.7	37.9	48.0	58.7	70.2	82.4	95.1
.92	2.33	7.02	13.2	20.5	28.8	38.0	48.1	58.8	70.3	82.5	95.3
.93	2.37	7.08	13.2	20.6	28.9	38.1	48.2	59.0	70.4	82.6	95.4
.94	2.41	7.13	13.3	20.6	29.0	38.2	48.3	59.1	70.6	82.7	95.5
.95	2.44	7.19	13.4	20.7	29.1	38.3	48.4	59.2	70.7	82.9	95.6
.96	2.48	7.24	13.4	20.8	29.2	38.4	48.5	59.3	70.8	83.0	95.8
.97	2.52	7.30	13.5	20.9	29.3	38.5	48.6	59.4	70.9	83.1	95.9
.98	2.56	7.36	13.6	21.0	29.3	38.6	48.7	59.5	71.0	83.2	96.0
.99	2.60	7.41	13.6	21.0	29.4	38.7	48.8	59.6	71.2	83.4	96.2
1.00	2.64	7.47	13.7	21.1	29.5	38.8	48.9	59.7	71.3	83.5	96.3

Note:—This table is applicable for use with broad-crested weirs exceeding 2 feet of crest width and for heads from 0.5 foot up to 1.5 or 2 times the breadth of weir crest.

DETERMINATION OF DISCHARGE OVER VARIOUS TYPES OF BROAD-CRESTED WEIRS.

From the weir experiments at the Cornell Hydraulic Laboratory, as outlined in United States Geological Survey Water-Supply Paper No. 200, Mr. E. C. Murphy has derived coefficients to be used in connection with a discharge table computed by Bazin's formula for sharp-crested weirs for determining the discharge over certain types of broad-crested weirs. Table IV gives the discharge per foot of length of crest by Bazin's formula for weirs having a height varying from 2 to 30 feet, and tables V, VI, and VII give the multipliers to be used with this table to give the discharge over broad-crested weirs. Example: Suppose the discharge is to be computed over a rectangular weir that is 10 feet long, 12 feet high, 6 feet crest width, and has an observed head of 2.4 feet.

Table IV shows that for a height (p) of 12 feet and a head (H) of 2.4, the discharge (Q) is 12.42 second-feet. Table V shows that for a height (p) of 12 feet, a crest width (c) of 6 feet, and head (H) of 2.4 feet the multiplier is 0.797. Hence, the discharge is $12.42 \times 0.797 \times 10 = 99.0$ second-feet.

Table IV.—Discharge in second-feet per foot of crest over sharp-crested rectangular weirs without end contractions.[a]

$$\text{Formula: } Q=\left(0.405+\frac{.0984}{H}\right)\left(1+0.55\frac{H^2}{(p+H)^2}\right) lH\sqrt{2gH}.$$

[H=head, in feet, P=height of weir, in feet].

H \ p	2	4	6	8	10	20	30
0.1	0.13	0.13	0.13	0.13	0.13	0.13	0.13
.2	.33	.33	.33	.33	.33	.33	.33
.3	.58	.58	.58	.58	.58	.58	.58
.4	.88	.88	.87	.87	.87	.87	.87
.5	1.23	1.21	1.21	1.21	1.21	1.20	1.20
.6	1.62	1.59	1.58	1.58	1.57	1.57	1.57
.7	2.04	1.99	1.98	1.98	1.97	1.97	1.97
.8	2.50	2.43	2.41	2.41	2.40	2.40	2.40
.9	3.00	2.90	2.88	2.86	2.86	2.85	2.85
1.0	3.53	3.40	3.36	3.35	3.34	3.33	3.33
1.1	4.10	3.93	3.88	3.86	3.85	3.84	3.83
1.2	4.69	4.48	4.42	4.40	4.38	4.36	4.36
1.3	5.32	5.07	4.99	4.96	4.94	4.92	4.91
1.4	5.99	5.68	5.58	5.55	5.52	5.49	5.48

[a] This table should not be used where water on the downstream side of the weir is above the level of the crest, nor unless air circulates freely between the overfalling sheet and the downstream face of the weir. If a vacuum forms under the falling sheet the discharge may be 5 per cent greater than given in this table.

Table IV.—Continued.

B \ p	2	4	6	8	10	20	30
1.5	6.69	6.30	6.20	6.16	6.13	6.08	6.07
1.6	7.40	6.97	6.84	6.78	6.75	6.69	6.68
1.7	8.15	7.66	7.50	7.43	7.39	7.33	7.31
1.8	8.93	8.37	8.18	8.09	8.05	7.98	7.96
1.9	9.74	9.11	8.89	8.79	8.74	8.65	8.63
2.0	10.58	9.87	9.62	9.51	9.44	9.34	9.32
2.1	11.44	10.65	10.37	10.24	10.17	10.05	10.02
2.2	12.33	11.46	11.14	10.99	10.91	10.78	10.75
2.3	13.25	12.29	11.93	11.77	11.67	11.52	11.48
2.4	14.20	13.15	12.75	12.56	12.45	12.28	12.24
2.5	15.18	14.03	13.59	13.37	13.25	13.06	13.02
2.6	16.17	14.92	14.44	14.20	14.07	13.85	13.80
2.7	17.19	15.84	15.31	15.05	14.90	14.65	14.60
2.8	18.23	16.79	16.21	15.92	15.76	15.48	15.42
2.9	19.29	17.75	17.12	16.81	16.63	16.32	16.25
3.0	20.38	18.74	18.06	17.71	17.52	17.18	17.10
3.1	21.50	19.74	19.01	18.64	18.42	18.05	17.96
3.2	22.64	20.77	19.98	19.58	19.34	18.93	18.83
3.3	23.80	21.82	20.98	20.54	20.28	19.83	19.72
3.4	24.98	22.89	21.99	21.52	21.24	20.75	20.63
3.5	26.20	23.98	23.01	22.51	22.22	21.69	21.55
3.6	27.42	25.09	24.06	23.52	23.20	22.62	22.48
3.7	28.67	26.23	25.13	24.55	24.21	23.58	23.43
3.8	29.94	27.38	26.22	25.60	25.23	24.56	24.39
3.9	31.23	28.55	27.32	26.66	26.27	25.54	25.37
4.0	32.54	29.74	28.45	27.74	27.32	26.55	26.35
4.1	33.87	30.96	29.59	28.84	28.39	27.56	27.34
4.2	35.22	32.18	30.75	29.96	29.48	28.59	28.35
4.3	36.59	33.43	31.92	31.09	30.58	29.63	29.38
4.4	37.99	34.70	33.12	32.24	31.70	30.68	30.42
4.5	39.40	35.98	34.33	33.40	32.83	31.74	31.47
4.6	40.83	37.29	35.56	34.58	33.98	32.82	32.53
4.7	42.28	38.61	36.80	35.78	35.14	33.92	33.61
4.8	43.75	39.96	38.07	37.00	36.32	35.04	34.70
4.9	45.23	41.32	39.35	38.23	37.52	36.17	35.80
5.0	46.73	42.69	40.65	39.48	38.74	37.21	36.91
5.1	48.25	44.09	41.96	40.73	39.97	38.45	38.03
5.2	49.79	45.50	43.29	42.01	41.20	39.61	39.17
5.3	51.36	46.93	44.64	43.30	42.45	40.78	40.31
5.4	52.94	48.38	46.00	44.60	43.71	41.96	41.47
5.5	54.54	49.85	47.38	45.93	45.00	43.16	42.64
5.6	56.15	51.34	48.79	47.27	46.31	44.38	43.83
5.7	57.78	52.83	50.19	48.62	47.62	45.60	45.02
5.8	59.42	54.34	51.62	49.99	48.94	46.83	46.22
5.9	61.09	55.88	53.07	51.38	50.29	48.08	47.44
6.0	62.77	57.43	54.53	52.78	51.64	49.34	48.67
6.1	64.46	59.00	56.00	54.20	53.02	50.61	49.91
6.2	66.18	60.58	57.50	55.63	54.40	51.90	51.16
6.3	67.91	62.18	59.01	57.07	55.80	53.20	52.42
6.4	69.65	63.79	60.53	58.53	57.22	54.50	53.70

Table IV.—Continued.

R \ p	2	4	6	8	10	20	30
6.5	71.42	65.42	62.07	60.01	58.65	55.82	54.98
6.6	73.19	67.07	63.63	61.50	60.09	57.16	56.27
6.7	74.99	68.74	65.20	63.00	61.55	58.50	57.58
6.8	76.80	70.42	66.78	64.53	63.02	59.96	58.90
6.9	78.62	72.11	68.38	66.06	64.50	61.23	60.22
7.0	80.46	73.82	70.00	67.60	66.00	62.61	61.56
7.1	82.32	75.55	71.63	69.17	67.52	64.00	62.91
7.2	84.18	77.29	73.28	70.74	69.04	65.40	64.27
7.3	86.07	79.04	74.94	72.34	70.58	66.81	65.64
7.4	87.97	80.81	76.61	73.94	72.14	68.24	67.02
7.5	89.89	82.60	78.30	75.56	73.70	69.68	68.41
7.6	91.82	84.40	80.01	77.19	75.28	71.13	69.81
7.7	93.76	86.22	81.73	78.84	76.88	72.59	71.23
7.8	95.72	88.05	83.46	80.50	78.48	74.06	72.65
7.9	97.70	89.90	85.21	82.18	80.11	75.55	74.09
8.0	99.68	91.75	86.97	83.87	81.74	77.04	75.53
8.1	101.69	93.63	88.75	85.57	83.39	78.55	76.98
8.2	103.70	95.51	90.54	87.29	85.25	80.06	78.44
8.3	105.73	97.42	92.34	89.02	86.72	81.59	79.92
8.4	107.78	99.34	94.16	90.76	88.41	83.13	81.40
8.5	109.84	101.27	96.00	92.52	90.11	84.69	82.90
8.6	111.91	103.21	97.84	94.29	91.82	86.25	84.41
8.7	113.99	105.17	99.70	96.07	93.55	87.82	85.92
8.8	116.09	107.14	101.57	97.87	95.28	89.40	87.44
8.9	118.20	109.13	103.46	99.68	97.04	91.00	88.98
9.0	120.33	111.13	105.36	101.50	98.80	92.61	90.52
9.1	122.47	113.15	107.28	103.34	100.58	94.23	92.08
9.2	124.62	115.18	109.21	105.19	102.37	95.86	93.65
9.3	126.79	117.22	111.15	107.06	104.17	97.49	95.22
9.4	128.97	119.27	113.10	108.93	105.99	99.14	96.80
9.5	131.16	121.34	115.07	110.82	107.82	100.80	98.40
9.6	133.36	123.42	117.05	112.72	109.65	102.48	100.00
9.7	135.58	125.51	119.04	114.64	111.50	104.16	101.62
9.8	137.82	127.63	121.05	116.57	113.37	105.85	103.25
9.9	140.06	129.74	123.07	118.51	115.25	107.56	104.88
10.0	142.31	131.87	125.10	120.46	117.14	109.27	106.52

Table V.—Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of rectangular cross-section of type a, fig. 24.

[p=Height of weir; c=width of crest; H=observed head, all in feet.]

H \ p.. / c..	4.6 2.6	4.6 6.6	11.25 .48	11.25 .93	11.25 1.65	11.25 3.17	11.25 5.88	11.25 8.98	11.25 12.24	11.25 16.30
0.5	....	....	0.821	0.792	0.806	0.792	0.799	0.801	0.786	0.790
1.0	0.765	0.706	.997	.899	.808	.795	.791	.794	.815	.790
1.5	.789	.709	1.00	.982	.878	.796	.796	.793	.814	.792
2.0	.814	.710	1.00	1.00	.906	.815	.797	.792	.797	.793
2.5	.835	.711	1.00	1.00	.985	.844	.797	.790	.796	.793
3.0	.857	.711	1.00	1.00	1.00	.870	.797	.788	.794	.791
3.5	.878	.712	1.00	1.00	1.00	.90	.812	.787	.794	.791
4.0	.899	.714	1.00	1.00	1.00	.93	.834	.786	.792	.789
5.0	.940	.716	1.00	1.00	1.00	.97	(a)	.78	.79	.78
6.0	.986	.718	1.00	1.00	1.00	.98	(a)	.78	.78	.78
7.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.78	.77
8.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.77	.77
9.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.77	.77
10.0	....	....	1.00	1.00	1.00	(a)	(a)	.77	.77	.77

[a] Value doubtful.

Table VI.—Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of trapezoidal cross-section of types b and c, fig. 24.

[p=Height of weir, in feet; c = width of crest, in feet; s = upstream slope; s' = downstream slope; H = observed head, in feet.]

	Type *b*, fig. 24.							Type *c*, fig. 24.	
p..........	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.65	11.25
c..........	.33	.66	.66	.66	.66	.33	.66	7.00	6.00
s..........	2:1	2:1	3:1	4:1	5:1	2:1	2:1	4.67:1	6:1
s'..........	0	0	0	0	0	5:1	2:1	....	....
H									
1.0........	1.137	1.048	1.066	1.039	1.009	1.095	1.071	1.042	1.060
1.5........	1.131	1.068	1.066	1.039	1.009	1.071	1.066	1.033	1.069
2.0........	1.120	1.080	1.061	1.033	1.005	1.044	1.053	1.024	1.054
2.5........	1.106	1.085	1.052	1.026	.997	1.024	1.047	1.012	1.012
3.0........	1.094	1.088	1.047	1.020	.991	1.009	1.047	.995	.985
3.5........	1.085	1.087	1.043	1.017	.988	1.003	1.050	.983	.979
4.0........	1.072	1.084	1.038	1.012	.984	1.014	1.052	.977	.976
4.5........	1.064	1.081	1.035	1.009	.980	1.023	1.055	.974	.973
5.0........	....	....	....	....	....	....	....	.97	.97
6.0........	....	....	....	....	....	....	....	.97	.96
7.0........	....	....	....	....	....	....	....	.97	.96
8.0........	....	....	....	....	....	....	....	.96	.95
9.0........	....	....	....	....	....	....	....	.96	.95
10.0........	....	....	....	....	....	....	....	.96	.95

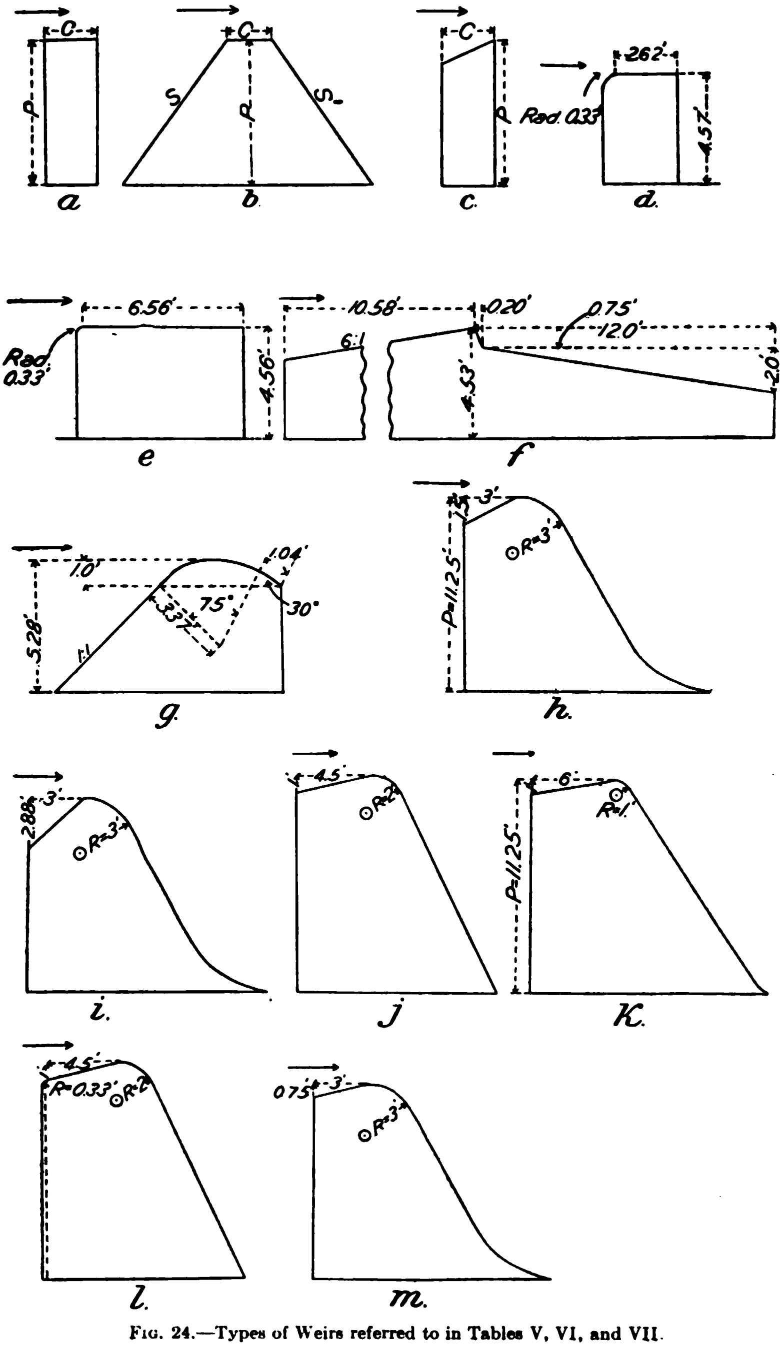


FIG. 24.—Types of Weirs referred to in Tables V, VI, and VII.

Table VII.—Multipliers to be used in connection with Table IV to obtain the discharge over broad-crested weirs of compound cross-section of types d to m inclusive, fig. 24.

[p=Height of weir, in feet; H=observed head, in feet.]

p	4.57	4.56	4.53	5.28	11.25	11.25	11.25	11.25	11.25	11.25
Type, fig. 24	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*
H										
0.5	....	....	....	....	0.941	0.924	0.933	0.962	0.971	0.947
1.0	0.842	0.836	0.929	0.976	1.089	1.033	.988	1.045	1.033	1.000
1.5	.866	.834	.950	.979	1.087	1.093	1.018	1.066	1.042	1.036
2.0	.888	.831	.953	.988	1.109	1.133	1.033	1.063	1.035	1.063
2.5	.906	.826	.947	1.000	1.118	1.153	1.045	1.020	1.033	1.085
3.0	.927	.822	.942	1.016	1.120	1.163	1.054	.997	1.045	1.096
3.5	.945	.817	.936	1.032	1.127	1.169	1.060	.994	1.054	1.108
4.0	.965	.812	.931	1.044	1.123	1.165	1.060	.991	1.057	1.110
5.0	1.00	.80	.92	1.05	1.11	1.16	1.05	.98	1.05	1.10
6.0	....	....	....	....	1.11	1.15	1.04	.98	1.04	1.10
7.0	....	....	....	....	1.10	1.14	1.04	.97	1.04	1.09
8.0	....	....	....	....	1.10	1.14	1.04	.97	1.03	1.09
9.0	....	....	....	....	1.09	1.14	1.03	.97	1.03	1.08
10.0	....	....	....	....	1.09	1.13	1.03	.97	1.03	1.08

Table VIII.—Three-halves powers for numbers from 0 to 12.

Units. / Hundredths.	0	1	2	3	4	5	6	7	8	9	10	11
0.00	0.0000	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829
.01	.0010	1.0150	2.8497	5.2222	8.0300	11.2139	14.7337	18.5600	22.6699	27.0450	31.6702	36.5326
.02	.0028	1.0302	2.8710	5.2482	8.0601	11.2475	14.7705	18.5997	22.7123	27.0890	31.7177	36.5824
.03	.0052	1.0453	2.8923	5.2743	8.0902	11.2811	14.8073	18.6394	22.7548	27.1351	31.7652	36.6322
.04	.0080	1.0606	2.9137	5.3004	8.1203	11.3148	14.8442	18.6792	22.7973	27.1802	31.8127	36.6820
.05	.0112	1.0759	2.9352	5.3266	8.1505	11.3485	14.8810	18.7190	22.8399	27.2253	31.8602	36.7319
.06	.0147	1.0913	2.9567	5.3528	8.1807	11.3822	14.9179	18.7589	22.8825	27.2705	31.9078	36.7818
.07	.0185	1.1068	2.9782	5.3791	8.2109	11.4160	14.9549	18.7988	22.9251	27.3156	31.9554	36.8317
.08	.0226	1.1224	2.9998	5.4054	8.2412	11.4497	14.9919	18.8387	22.9677	27.3608	32.0030	36.8816
.09	.0270	1.1380	3.0215	5.4317	8.2715	11.4836	15.0289	18.8786	23.0103	27.4060	32.0506	36.9315
0.10	0.0316	1.1537	3.0432	5.4581	8.3019	11.5174	15.0659	18.9185	23.0530	27.4512	32.0983	36.9815
.11	.0365	1.1695	3.0650	5.4845	8.3323	11.5513	15.1030	18.9585	23.0957	27.4965	32.1460	37.0315
.12	.0416	1.1853	3.0868	5.5110	8.3627	11.5852	15.1400	18.9985	23.1384	27.5418	32.1937	37.0815
.13	.0469	1.2012	3.1086	5.5375	8.3932	11.6192	15.1772	19.0386	23.1812	27.5871	32.2414	37.1315
.14	.0524	1.2172	3.1306	5.5641	8.4237	11.6532	15.2143	19.0786	23.2240	27.6324	32.2892	37.1816
.15	.0581	1.2332	3.1525	5.5907	8.4542	11.6872	15.2515	19.1187	23.2668	27.6778	32.3370	37.2317
.16	.0640	1.2494	3.1745	5.6173	8.4848	11.7213	15.2887	19.1589	23.3096	27.7232	32.3848	37.2817
.17	.0701	1.2656	3.1966	5.6440	8.5154	11.7554	15.3260	19.1990	23.3525	27.7686	32.4326	37.3319
.18	.0764	1.2818	3.2187	5.6708	8.5460	11.7895	15.3632	19.2392	23.3954	27.8140	32.4804	37.3820
.19	.0828	1.2981	3.2409	5.6975	8.5767	11.8236	15.4005	19.2794	23.4383	27.8595	32.5283	37.4322

Table VIII.—Continued.

Hundredths \ Units	0	1	2	3	4	5	6	7	8	9	10	11
0.20	0.0894	1.3145	3.2631	5.7243	8.6074	11.8578	15.4879	19.3196	23.4812	27.9050	32.5762	37.4824
.21	.0962	1.3310	3.2854	5.7512	8.6382	11.8920	15.4752	19.3599	23.5242	27.9514	32.6241	37.5326
.22	.1032	1.3475	3.3077	5.7781	8.6690	11.9263	15.5126	19.4002	23.5672	27.9960	32.6720	37.5828
.23	.1103	1.3641	3.3301	5.8050	8.6998	11.9606	15.5501	19.4405	23.6102	28.0416	32.7200	37.6331
.24	.1176	1.3808	3.3525	5.8320	8.7307	11.9949	15.5866	19.4808	23.6533	28.0872	32.7680	37.6833
.25	.1250	1.3975	3.3750	5.8590	8.7616	12.0293	15.6250	19.5212	23.6963	28.1328	32.8160	37.7336
.26	.1326	1.4144	3.3975	5.8861	8.7925	12.0636	15.6616	19.5576	23.7394	28.1784	32.8640	37.7840
.27	.1403	1.4312	3.4201	5.9132	8.8235	12.0981	15.7001	19.6021	23.7825	28.2241	32.9121	37.8343
.28	.1482	1.4482	3.4427	5.9403	8.8545	12.1325	15.7376	19.6425	23.8257	28.2698	32.9600	37.8847
.29	.1562	1.4652	3.4654	5.9675	8.8856	12.1670	15.7752	19.6830	23.8689	28.3155	33.0083	37.9351
0.30	0.1643	1.4822	3.4881	5.9947	8.9167	12.2015	15.8129	19.7235	23.9121	28.3612	33.0564	37.9855
.31	.1726	1.4994	3.5109	6.0220	8.9478	12.2361	15.8505	19.7641	23.9553	28.4069	33.1046	38.0359
.32	.1810	1.5166	3.5337	6.0493	8.9790	12.2706	15.8882	19.8046	23.9986	28.4527	33.1527	38.0864
.33	.1896	1.5338	3.5566	6.0767	9.0102	12.3053	15.9260	19.8452	24.0418	28.4985	33.2009	38.1369
.34	.1983	1.5512	3.5796	6.1041	9.0414	12.3399	15.9637	19.8858	24.0851	28.5444	33.2492	38.1874
.35	.2071	1.5686	3.6025	6.1315	9.0726	12.3746	16.0015	19.9265	24.1285	28.5902	33.2974	38.2379
.36	.2160	1.5860	3.6255	6.1590	9.1040	12.4093	16.0393	19.9672	24.1718	28.6361	33.3457	38.2884
.37	.2251	1.6035	3.6486	6.1865	9.1353	12.4440	16.0772	20.0079	24.2152	28.6820	33.3940	38.3390
.38	.2342	1.6211	3.6717	6.2141	9.1667	12.4788	16.1150	20.0486	24.2586	28.7279	33.4423	38.3896
.39	.2436	1.6388	3.6949	6.2417	9.1981	12.5136	16.1529	20.0894	24.3021	28.7739	33.4906	38.4402
0.40	0.2530	1.6565	3.7181	6.2693	9.2295	12.5485	16.1909	20.1302	24.3455	28.8199	33.5390	38.4908
.41	.2625	1.6743	3.7413	6.2970	9.2610	12.5833	16.2288	20.1710	24.3890	28.8659	33.5874	38.5415
.42	.2722	1.6921	3.7646	6.3247	9.2925	12.6182	16.2668	20.2118	24.4325	28.9119	33.6358	38.5922
.43	.282[illegible]	1.7100	3.7880	6.3525	9.3241	12.6532	16.3048	20.2527	24.4761	28.9579	33.6842	38.6429
.44	.2919	1.7280	3.8114	6.3803	9.3557	12.6882	16.3429	20.2936	24.5196	29.0040	33.7327	38.6936
.45	.3019	1.7460	3.8349	6.4081	9.3873	12.7232	16.3810	20.3345	24.5632	29.0501	33.7811	38.7443
.46	.3120	1.7641	3.8584	6.4360	9.4189	12.7582	16.4191	20.3755	24.6068	29.0962	33.8297	38.7951
.47	.3222	1.7823	3.8819	6.4639	9.4506	12.7933	16.4572	20.4165	24.6505	29.1424	33.8782	38.8459
.48	.3325	1.8005	3.9055	6.4919	9.4824	12.8284	16.4954	20.4575	24.6941	29.1885	33.9267	38.8967
.49	.3430	1.8188	3.9292	6.5199	9.5141	12.8635	16.5336	20.4985	24.7378	29.2347	33.9753	38.9475
0.50	0.3536	1.8371	3.9529	6.5479	9.5459	12.8986	16.5718	20.5396	24.7815	29.2810	34.0239	38.9984
.51	.3642	1.8555	3.9766	6.5760	9.5778	12.9338	16.6101	20.5807	24.8253	29.3272	34.0725	39.0493
.52	.3750	1.8740	4.0004	6.6041	9.6097	12.9691	16.6484	20.6218	24.8691	29.3735	34.1211	39.1002
.53	.3858	1.8925	4.0242	6.6323	9.6416	13.0043	16.6867	20.6630	24.9129	29.4198	34.1698	39.1511
.54	.3968	1.9111	4.0481	6.6605	9.6735	13.0396	16.7250	20.7041	24.9567	29.4661	34.2185	39.2020
.55	.4079	1.9297	4.0720	6.6887	9.7055	13.0749	16.7634	20.7453	25.0005	29.5124	34.2672	39.2530
.56	.4191	1.9484	4.0960	6.7170	9.7375	13.1103	16.8018	20.7866	25.0444	29.5588	34.3159	39.3040
.57	.4303	1.9672	4.1200	6.7453	9.7695	13.1457	16.8402	20.8278	25.0883	29.6052	34.3647	39.3550
.58	.4417	1.9860	4.1441	6.7737	9.8016	13.1811	16.8787	20.8691	25.1322	29.6516	34.4135	39.4060
.59	.4532	2.0049	4.1682	6.8021	9.8337	13.2165	16.9172	20.9104	25.1762	29.6980	34.4623	39.4571
0.60	0.4648	2.0238	4.1924	6.8305	9.8659	13.2520	16.9557	20.9518	25.2202	29.7445	34.5111	39.5082
.61	.4764	2.0429	4.2166	6.8590	9.8981	13.2875	16.9943	20.9931	25.2642	29.7910	34.5599	39.5593
.62	.4882	2.0619	4.2408	6.8875	9.9303	13.3231	17.0328	21.0345	25.3082	29.8375	34.6088	39.6104
.63	.5000	2.0810	4.2651	6.9161	9.9626	13.3587	17.0714	21.0759	25.3522	29.8841	34.6577	39.6615
.64	.5120	2.1002	4.2895	6.9447	9.9949	13.3943	17.1101	21.1174	25.3963	29.9306	34.7066	39.7127
.65	.5240	2.1195	4.3139	6.9733	10.0272	13.4299	17.1488	21.1589	25.4404	29.9772	34.7557	39.7639
.66	.5362	2.1388	4.3383	7.0020	10.0596	13.4656	17.1874	21.2004	25.4845	30.0238	34.8045	39.8151
.67	.5484	2.1581	4.3628	7.0307	10.0920	13.5013	17.2172	21.2419	25.5287	30.0704	34.8535	39.8663
.68	.5607	2.1775	4.3874	7.0595	10.1244	13.5370	17.2649	21.2834	25.5729	30.1171	34.9025	39.9176
.69	.5732	2.1970	4.4119	7.0883	10.1569	13.5728	17.3037	21.3250	25.6171	30.1638	34.9516	39.9689

Table VIII.—Continued.

Hundredths. \ Units.	0	1	2	3	4	5	6	7	8	9	10	11
0.70	0.5857	2.2165	4.4366	7.1171	10.1894	13.6086	17.3425	21.3666	25.6613	30.2105	35.0006	40.0202
.71	.5983	2.2361	4.4612	7.1460	10.2214	13.6444	17.3814	21.4083	25.7056	30.2572	35.0497	40.0715
.72	.6109	2.2558	4.4859	7.1749	10.2545	13.6803	17.4202	21.4499	25.7499	30.3040	35.0988	40.1228
.73	.6237	2.2755	4.5107	7.2038	10.2871	13.7161	17.4591	21.4916	25.7942	30.3507	35.1479	40.1742
.74	.6366	2.2952	4.5355	7.2328	10.3197	13.7521	17.4981	21.5333	25.8395	30.3975	35.1971	40.2256
.75	.6495	2.3150	4.5604	7.2618	10.3524	13.7880	17.5370	21.5751	25.8828	30.4444	35.2462	40.2770
.76	.6626	2.3349	4.5853	7.2909	10.3851	13.8240	17.5760	21.6169	25.9272	30.4912	35.2954	40.3284
.77	.6757	2.3548	4.6102	7.3200	10.4178	13.8600	17.6150	21.6587	25.9716	30.5381	35.3446	40.3798
.78	.6889	2.3748	4.6352	7.3492	10.4506	13.8961	17.6541	21.7005	26.0161	30.5850	35.3939	40.4313
.79	.7022	2.3949	4.6602	7.3783	10.4834	13.9321	17.6981	21.7423	26.0605	30.6319	35.4431	40.4828
0.80	0.7155	2.4150	4.6853	7.4076	10.5163	13.9682	17.7322	21.7842	26.1050	30.6789	35.4924	40.5343
.81	.7290	2.4351	4.7104	7.4368	10.5492	14.0044	17.7714	21.8261	26.1495	30.7258	35.5417	40.5859
.82	.7425	2.4553	4.7356	7.4661	10.5812	14.0406	17.8105	21.8681	26.1941	30.7728	35.5911	40.6374
.83	.7562	2.4756	4.7608	7.4955	10.6150	14.0768	17.8507	21.9100	26.2386	30.8198	35.6404	40.6890
.84	.7699	2.4959	4.7861	7.5248	10.6480	14.1130	17.8889	21.9520	26.2832	30.8669	35.6898	40.7406
.85	.7837	2.5163	4.8114	7.5542	10.6810	14.1493	17.9282	21.9940	26.3278	30.9139	35.7392	40.7922
.86	.7975	2.5367	4.8367	7.5837	10.7141	14.1856	17.9674	22.0361	26.3725	30.9610	35.7886	40.8439
.87	.8115	2.5572	4.8621	7.6132	10.7472	14.2219	18.0067	22.0781	26.4171	31.0081	35.8380	40.8955
.88	.8255	2.5777	4.8875	7.6427	10.7803	14.2582	18.0461	22.1202	26.4618	31.0553	35.8875	40.9472
.89	.8396	2.5983	4.9130	7.6723	10.8134	14.2946	18.0854	22.1623	26.5065	31.1024	35.9370	40.9989
0.90	0.8538	2.6190	4.9385	7.7019	10.8466	14.3311	18.1248	22.2045	26.5523	31.1496	35.9865	41.0507
.91	.8681	2.6397	4.9641	7.7315	10.8798	14.3675	18.1642	22.2467	26.5960	31.1968	36.0360	41.1024
.92	.8824	2.6604	4.9897	7.7702	10.9131	14.4040	18.2037	22.2889	26.6408	31.2441	36.0856	41.1542
.93	.8969	2.6812	5.0154	7.7909	10.9464	14.4405	18.2432	22.3811	26.6856	31.2913	36.1352	41.2060
.94	.9114	2.7021	5.0411	7.8207	10.9797	14.4770	18.2827	22.3733	26.7305	31.3386	36.1848	41.2578
.95	.9259	2.7230	5.0668	7.8505	11.0131	14.5136	18.3222	22.4156	26.7753	31.3850	36.2344	41.3097
.96	.9406	2.7440	5.0926	7.8808	11.0464	14.5502	18.3617	22.4579	26.8202	31.4332	36.2841	41.3615
.97	.9553	2.7650	5.1184	7.9102	11.0799	14.5869	18.4013	22.5003	26.8651	31.4806	36.3337	41.4134
.98	.9702	2.7861	5.1443	7.9401	11.1133	14.6235	18.4409	22.5426	26.9100	31.5280	36.3834	41.4653
.99	.9850	2.8072	5.1702	7.9700	11.1468	14.6602	18.4806	22.5850	26.9550	31.5754	36.4331	41.5173
1.00	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829	41.5692

Table IX.—For converting discharge in second-feet per square mile into run-off in depth in inches over the area.

Second-feet per square mile.	Period in days.				
	1	28	29	30	31
	Inches.	*Inches.*	*Inches.*	*Inches.*	*Inches.*
1	.03719	1.041	1.079	1.116	1.153
2	.07438	2.083	2.157	2.231	2.306
3	.11157	3.124	3.236	3.347	3.459
4	.14876	4.165	4.314	4.463	4.612
5	.18595	5.207	5.393	5.579	5.764
6	.22314	6.248	6.471	6.694	6.917
7	.26033	7.289	7.550	7.810	8.070
8	.29752	8.331	8.628	8.926	9.223
9	.33471	9.372	9.707	10.041	10.376

NOTE.—For partial month multiply the values for one day by the number of days.

Table X.—For converting discharge in second-feet into run-off in acre-feet.

Second-feet.	Period in days.				
	1	28	29	30	31
	Acre-ft.	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*	*Acre-ft.*
1	1.983	55.54	57.52	59.50	61.49
2	3.967	111.1	115.0	119.0	123.0
3	5.950	166.6	172.6	178.5	184.5
4	7.934	222.1	230.1	238.0	246.0
5	9.917	277.7	287.6	297.5	307.4
6	11.90	333.2	345.1	357.0	368.9
7	13.88	388.8	402.6	416.5	430.4
8	15.87	444.3	460.2	476.0	491.9
9	17.85	499.8	517.7	535.5	553.4

NOTE.—For partial month multiply values for one day by the number of days.

Table XI.—For converting discharge in second-feet per day into run-off in millions of gallons.

1 second-foot, or 7.4805 gallons per second for 1 day, or 86,400 seconds = 646,300 gallons.

Tens.	Units.									
	0	1	2	3	4	5	6	7	8	9
0	....	0.65	1.29	1.94	2.59	3.23	3.88	4.52	5.17	5.82
1	6.46	7.11	7.76	8.40	9.05	9.69	10.34	10.99	11.63	12.28
2	12.93	13.57	14.22	14.87	15.51	16.16	16.80	17.45	18.10	18.74
3	19.39	20.04	20.68	21.33	21.97	22.62	23.27	23.91	24.56	25.21
4	25.85	26.50	27.15	27.79	28.44	29.08	29.73	30.38	31.02	31.67
5	32.32	32.96	33.61	34.25	34.90	35.55	36.19	36.84	37.49	38.13
6	38.78	39.43	40.17	40.72	41.36	42.01	42.66	43.30	43.95	44.60
7	45.24	45.89	46.53	47.18	47.83	48.47	49.12	49.77	50.41	51.06
8	51.71	52.35	53.00	53.64	54.29	54.94	55.58	56.23	56.88	57.52
9	58.17	58.81	59.46	60.11	60.75	61.40	62.05	62.69	63.34	63.99

Table XII.—For converting run-off in millions of gallons into discharge in second-feet per day.

1 million gallons per 24 hours $= \frac{231,000,000}{1,728 \times 86,400}$ cubic feet per second, or 1.547 second feet.

Tens.	Units.									
	0	1	2	3	4	5	6	7	8	9
0	....	1.55	3.09	4.64	6.19	7.74	9.28	10.83	12.38	13.93
1	15.47	17.02	18.57	20.11	21.66	23.21	24.76	26.30	27.85	29.40
2	30.94	32.49	34.04	35.59	37.13	38.68	40.23	41.78	43.32	44.87
3	46.42	47.96	49.51	51.06	52.61	54.15	55.70	57.25	58.79	60.34
4	61.89	63.44	64.98	66.53	68.08	69.63	71.17	72.72	74.27	75.81
5	77.36	78.91	80.46	82.00	83.55	85.10	86.64	88.19	89.74	91.29
6	92.83	94.38	95.93	97.48	99.02	100.57	102.12	103.66	105.21	106.76
7	108.31	109.85	111.40	112.95	114.49	116.04	117.59	119.14	120.68	122.23
8	123.78	125.33	126.87	128.42	129.97	131.51	133.06	134.61	136.16	137.70
9	139.25	140.80	142.34	143.89	145.44	146.99	148.53	150.08	151.63	153.18

Table XIII.—For converting run-off in acre-feet into run-off in million gallons.

1 acre-foot = 43,560 cubic feet $= \frac{43,560 \times 1,728}{231}$, or $\frac{75,271,680}{231}$ or 325,850 gallons.

Tens.	Units. 0	1	2	3	4	5	6	7	8	9
0	....	0.326	0.652	0.978	1.303	1.629	1.955	2.281	2.607	2.933
1	3.258	3.584	3.910	4.236	4.562	4.888	5.214	5.540	5.865	6.191
2	6.517	5.843	7.169	7.495	7.820	8.146	8.472	8.798	9.124	9.450
3	9.776	10.101	10.427	10.753	11.079	11.405	11.731	12.056	12.382	12.708
4	13.034	13.360	13.686	14.012	14.337	14.663	14.989	15.315	15.641	15.967
5	16.293	16.618	16.944	17.270	17.596	17.922	18.248	18.574	18.899	19.225
6	19.551	19.877	20.203	20.529	20.854	21.180	21.506	21.832	22.158	22.484
7	22.810	23.135	23.461	23.787	24.113	24.439	24.765	25.090	25.416	25.742
8	26.068	26.394	26.720	27.046	27.372	27.697	28.023	28.349	28.675	29.001
9	29.327	29.652	29.978	30.304	30.630	30.956	31.282	31.608	31.933	32.259

Table XIV.—For converting run-off in million gallons into run-off in acre-feet.

One million United States liquid gallons or 231 million cubic inches = 133,680.555 cubic feet, or $\frac{133,680}{43,560} = 3.0689$ acre-feet.

Tens.	Units. 0	1	2	3	4	5	6	7	8	9
0	....	3.07	6.14	9.21	12.28	15.34	18.41	21.48	24.55	27.62
1	30.69	33.76	36.83	39.90	42.96	46.03	49.10	52.17	55.24	58.31
2	61.38	64.45	67.52	70.58	73.65	76.72	79.79	82.86	85.93	89.00
3	92.07	95.14	98.20	101.27	104.34	107.41	110.48	113.55	116.62	119.69
4	122.76	125.82	128.89	131.96	135.03	138.10	141.17	144.24	147.31	150.38
5	153.44	156.51	159.58	162.65	165.72	168.79	171.86	174.93	178.00	181.06
6	184.13	187.20	190.27	193.34	196.41	199.48	202.55	205.62	208.68	211.75
7	214.82	217.89	220.96	224.03	227.10	230.17	233.24	236.30	239.37	242.44
8	245.51	248.58	251.65	254.72	257.79	260.86	263.92	266.99	270.06	273.13
9	276.20	279.27	282.34	285.41	288.48	291.54	294.61	297.68	300.75	303.82

Table XV.—Values of c for use in the Chezy formula $V = c\sqrt{Rs}$.

Slope.	R.	n .020	n .025	n .030	n .035	n .040	n .045	n .050	n .055	n .060
.0001	3.28	91	73	60	52	46	40	36	33	30
	10	111	92	78	69	62	55	50	46	42
	20	122	102	89	79	71	65	60	55	51
	50	134	114	100	91	83	76	71	67	63
	100	140	121	108	98	91	84	79	74	70
.0002	10	108	89	76	67	60	53	49	45	41
	20	117	98	85	76	68	61	57	53	49
	50	126	108	94	85	78	71	66	62	58
	100	131	113	99	90	83	77	72	68	64
.0004	10	107	88	75	66	59	53	48	44	41
	20	115	96	83	73	66	60	55	51	48
	50	123	104	91	82	75	68	63	59	56
	100	127	108	96	87	80	73	68	64	61
.0010	10	105	87	74	65	58	52	47	44	40
	20	113	94	81	72	65	59	54	50	47
	50	120	101	89	79	72	66	61	57	54
	100	124	105	94	85	77	71	66	62	59
.010	10	105	86	74	65	58	51	47	43	40
	20	112	93	80	71	64	58	53	49	46
	50	119	100	87	78	71	65	60	56	53
	100	122	104	91	82	75	69	65	61	58

NOTE.—For $R = 3.28$ feet, n constant, c is constant for all values of slope. For slopes greater than 0.01, or fall of 52.8 feet per mile, c remains nearly constant.

Table XVI.—Square roots of numbers ($\sqrt{R}$ $\sqrt{s}$) for use in Kutter's formula. See Pl. VI, Chapter IV.

R	$\sqrt{R}$	R	$\sqrt{R}$	R	$\sqrt{R}$	R	$\sqrt{R}$	R	$\sqrt{R}$	s	$\sqrt{s}$
0.05	0.224	3.05	1.746	6.05	2.460	9.05	3.008	20.25	4.500	.00002	.00447
0.10	0.316	3.10	1.761	6.10	2.470	9.10	3.017	20.50	4.528	.000025	.005
0.15	0.387	3.15	1.775	6.15	2.480	9.15	3.025	20.75	4.555	.0000275	.00524
0.20	0.447	3.20	1.789	6.20	2.490	9.20	3.033	21.00	4.583	.00003	.00548
0.25	0.500	3.25	1.803	6.25	2.500	9.25	3.041	21.25	4.610	.000035	.00592
0.30	0.548	3.30	1.817	6.30	2.510	9.30	3.050	21.50	4.637	.00004	.00632
0.35	0.592	3.35	1.830	6.35	2.520	9.35	3.058	21.75	4.664	.000045	.00671
0.40	0.632	3.40	1.844	6.40	2.530	9.40	3.066	22.00	4.690	.00005	.00707
0.45	0.671	3.45	1.857	6.45	2.540	9.45	3.074	22.25	4.717	.00006	.00775
0.50	0.707	3.50	1.871	6.50	2.550	9.50	3.082	22.50	4.743	.00007	.00837
0.55	0.742	3.55	1.884	6.55	2.559	9.55	3.090	22.75	4.770	.00008	.00894
0.60	0.775	3.60	1.897	6.60	2.569	9.60	3.098	23.00	4.796	.00009	.00949
0.65	0.806	3.65	1.910	6.65	2.579	9.65	3.106	23.25	4.822	.0001	.01
0.70	0.837	3.70	1.924	6.70	2.588	9.70	3.114	23.50	4.848	.00012	.0110
0.75	0.866	3.75	1.936	6.75	2.598	9.75	3.122	23.75	4.873	.00014	.0118
0.80	0.894	3.80	1.949	6.80	2.608	9.80	3.130	24.00	4.899	.00016	.0126
0.85	0.922	3.85	1.962	6.85	2.617	9.85	3.138	24.25	4.924	.00018	.0134
0.90	0.949	3.90	1.975	6.90	2.627	9.90	3.146	24.50	4.950	.0002	.0141
0.95	0.975	3.95	1.987	6.95	2.636	9.95	3.154	24.75	4.975	.00025	.0158
1.00	1.000	4.00	2.000	7.00	2.646	10.00	3.162	25.00	5.000	.0003	.0173
1.05	1.025	4.05	2.012	7.05	2.655	10.25	3.202	25.25	5.025	.00035	.0187
1.10	1.049	4.10	2.025	7.10	2.665	10.50	3.240	25.50	5.050	.0004	.02
1.15	1.072	4.15	2.037	7.15	2.674	10.75	3.279	25.75	5.074	.0005	.0224
1.20	1.095	4.20	2.049	7.20	2.683	11.00	3.317	26.00	5.099	.0006	.0245
1.25	1.118	4.25	2.062	7.25	2.693	11.25	3.354	26.25	5.123	.0007	.0265
1.30	1.140	4.30	2.074	7.30	2.702	11.50	3.391	26.50	5.148	.0008	.0283
1.35	1.162	4.35	2.086	7.35	2.711	11.75	3.428	26.75	5.172	.0009	.03
1.40	1.183	4.40	2.098	7.40	2.720	12.00	3.464	27.00	5.196	.001	.0316
1.45	1.204	4.45	2.110	7.45	2.729	12.25	3.500	27.25	5.220	.0012	.0346
1.50	1.225	4.50	2.121	7.50	2.739	12.50	3.536	27.50	5.244	.0014	.0374
1.55	1.245	4.55	2.133	7.55	2.748	12.75	3.571	27.75	5.268	.0016	.04
1.60	1.265	4.60	2.145	7.60	2.757	13.00	3.606	28.00	5.292	.0018	.0424
1.65	1.285	4.65	2.156	7.65	2.766	13.25	3.640	28.25	5.315	.002	.0447
1.70	1.304	4.70	2.168	7.70	2.775	13.50	3.674	28.50	5.339	.003	.0548
1.75	1.323	4.75	2.179	7.75	2.784	13.75	3.708	28.75	5.362	.004	.0632
1.80	1.342	4.80	2.191	7.80	2.793	14.00	3.742	29.00	5.385	.005	.0707
1.85	1.360	4.85	2.202	7.85	2.802	14.25	3.775	29.25	5.408	....	......
1.90	1.378	4.90	2.214	7.90	2.811	14.50	3.808	29.50	5.431	....	......
1.95	1.396	4.95	2.225	7.95	2.820	14.75	3.841	29.75	5.454	....	......
2.00	1.414	5.00	2.236	8.00	2.828	15.00	3.873	30.00	5.477	....	......
2.05	1.432	5.05	2.247	8.05	2.837	15.25	3.905	30.25	5.500	....	......
2.10	1.449	5.10	2.258	8.10	2.846	15.50	3.937	30.50	5.523	....	......
2.15	1.466	5.15	2.269	8.15	2.855	15.75	3.969	30.75	5.545	....	......
2.20	1.483	5.20	2.280	8.20	2.864	16.00	4.000	31.00	5.568	....	......
2.25	1.500	5.25	2.291	8.25	2.872	16.25	4.031	31.25	5.590	....	......
2.30	1.517	5.30	2.302	8.30	2.881	16.50	4.062	31.50	5.612	....	......
2.35	1.533	5.35	2.313	8.35	2.890	16.75	4.093	31.75	5.635	....	......
2.40	1.549	5.40	2.324	8.40	2.898	17.00	4.123	32.00	5.657	....	......
2.45	1.565	5.45	2.335	8.45	2.907	17.25	4.153	32.25	5.679	....	......
2.50	1.581	5.50	2.345	8.50	2.915	17.50	4.183	32.50	5.701	....	......
2.55	1.597	5.55	2.356	8.55	2.924	17.75	4.213	32.75	5.723	....	......
2.60	1.612	5.60	2.366	8.60	2.933	18.00	4.243	33.00	5.745	....	......
2.65	1.628	5.65	2.377	8.65	2.941	18.25	4.272	33.25	5.766	....	......
2.70	1.643	5.70	2.387	8.70	2.950	18.50	4.301	33.50	5.788	....	......
2.75	1.658	5.75	2.398	8.75	2.958	18.75	4.330	33.75	5.809	....	......
2.80	1.673	5.80	2.408	8.80	2.966	19.00	4.359	34.00	5.831	....	......
2.85	1.688	5.85	2.419	8.85	2.975	19.25	4.387	34.25	5.852	....	......
2.90	1.703	5.90	2.429	8.90	2.983	19.50	4.416	34.50	5.874	....	......
2.95	1.718	5.95	2.439	8.95	2.992	19.75	4.444	34.75	5.895	....	......
3.00	1.732	6.00	2.449	9.00	3.000	20.00	4.472	35.00	5.916	....	......

Table XVII.—Convenient equivalents.

1 second-foot equals 40 California miner's inches. (Law of March 23, 1901.)

1 second-foot equals 38.4 Colorado miner's inches.

1 second-foot equals 40 Arizona miner's inches.

1 second-foot equals 7.48 United States gallons per second; equals 448.8 gallons per minute; equals 646,272 gallons for one day.

1 second-foot equals 6.23 British imperial gallons per second.

1 second-foot for one year covers one square mile 1.131 feet deep; 13.57 inches deep.

1 second-foot for one year equals 31,536,000 cubic feet.

1 second-foot equals about 1 acre-inch per hour.

1 second-foot falling 10 feet equals 1.136 horsepower.

100 California miner's inches equal 15.7 United States gallons per second.

100 California miner's inches equal 96.0 Colorado miner's inches.

100 California miner's inches for one day equal 4.96 acre-feet.

100 Colorado miner's inches equal 2.60 second-feet.

100 Colorado miner's inches equal 19.5 United States gallons per second.

100 Colorado miner's inches equal 130 California miner's inches.

100 Colorado miner's inches for one day equal 5.17 acre-feet.

100 United States gallons per minute equal 0.223 second-feet.

100 United States gallons per minute for one day equal 0.442 acre-feet.

1,000,000 United States gallons per day equal 1.55 second-feet.

1,000,000 United States gallons equal 3.07 acre-feet.

1,000,000 cubic feet equal 22.95 acre-feet.

1 acre-foot equals 325,850 gallons.

1 inch deep on 1 square mile equals 2,323,200 cubic feet.

1 inch deep on 1 square mile equals 0.0737 second-foot per year.

1 inch equals 2.54 centimeters.

1 foot equals 0.3048 meter.

1 yard equals 0.9144 meter.

1 mile equals 1.60935 kilometers.

1 mile equals 1,760 yards; equals 5,280 feet; equals 63,360 inches.

1 square yard equals 0.836 square meter.

1 acre equals 0.4047 hectare.

1 acre equals 43,560 square feet; equals 4,840 square yards.

1 acre equals 209 feet square, nearly.

1 square mile equals 259 hectares.

1 square mile equals 2.59 square kilometers.

1 cubic foot equals 0.0283 cubic meter.

1 cubic foot equals 7.48 gallons; equals 0.804 bushel.

1 cubic foot of water weighs 62.5 pounds.

1 cubic yard equals 0.7646 cubic meter.

1 gallon equals 3.7854 liters.

1 gallon equals 8.36 pounds of water.

1 gallon equals 231 cubic inches (liquid measure).

1 pound equals 0.4536 kilogram.

1 avoirdupois pound equals 7,000 grains.

1 troy pound equals 5,760 grams.

1 meter equals 39.37 inches. Log. 1.5951654.

1 meter equals 3.280833 feet. Log. 0.5159842.

1 meter equals 1.093611 yards. Log. 0.0388629.

1 kilometer equals 3,281 feet; equals five-eighths mile, nearly.

1 square meter equals 10.764 square feet; equals 1.196 square yards.

1 hectare equals 2.471 acres.

1 cubic meter equals 35.314 cubic feet; equals 1.308 cubic yards.

1 liter equals 1.0567 quarts.

1 gram equals 15.43 grains.

1 kilogram equals 2.2046 pounds.

1 tonneau equals 2,204.6 pounds.

1 foot per second equals 1.097 kilometers per hour.

1 foot per second equals 0.68 mile per hour.

1 cubic meter per minute equals 0.5886 second-foot.

1 atmosphere equals 15 pounds per square inch; equals 1 ton per square foot; equals 1 kilogram per square centimeter.

Acceleration of gravity equals 32.16 feet per second every second.

1 horsepower equals 550 foot-pounds per second.

1 horsepower equals 760 kilogram-meters per second.

1 horsepower equals 746 watts.

1 horsepower equals 1 second-foot falling 8.80 feet.

$1\frac{1}{3}$ horsepowers equal about 1 kilowatt.

To calculate water power quickly: $\frac{\text{Sec.-ft.} \times \text{fall in feet}}{11}$ = Net horsepower on water wheel, realizing 80 per cent of the theoretical power.

To change miles to inches on map:

Scale 1:125000, 1 mile = 0.50688 inch.
Scale 1:90000, 1 mile = 0.70400 inch.
Scale 1:62500, 1 mile = 1.01376 inches.
Scale 1:45000, 1 mile = 1.40800 inches.

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